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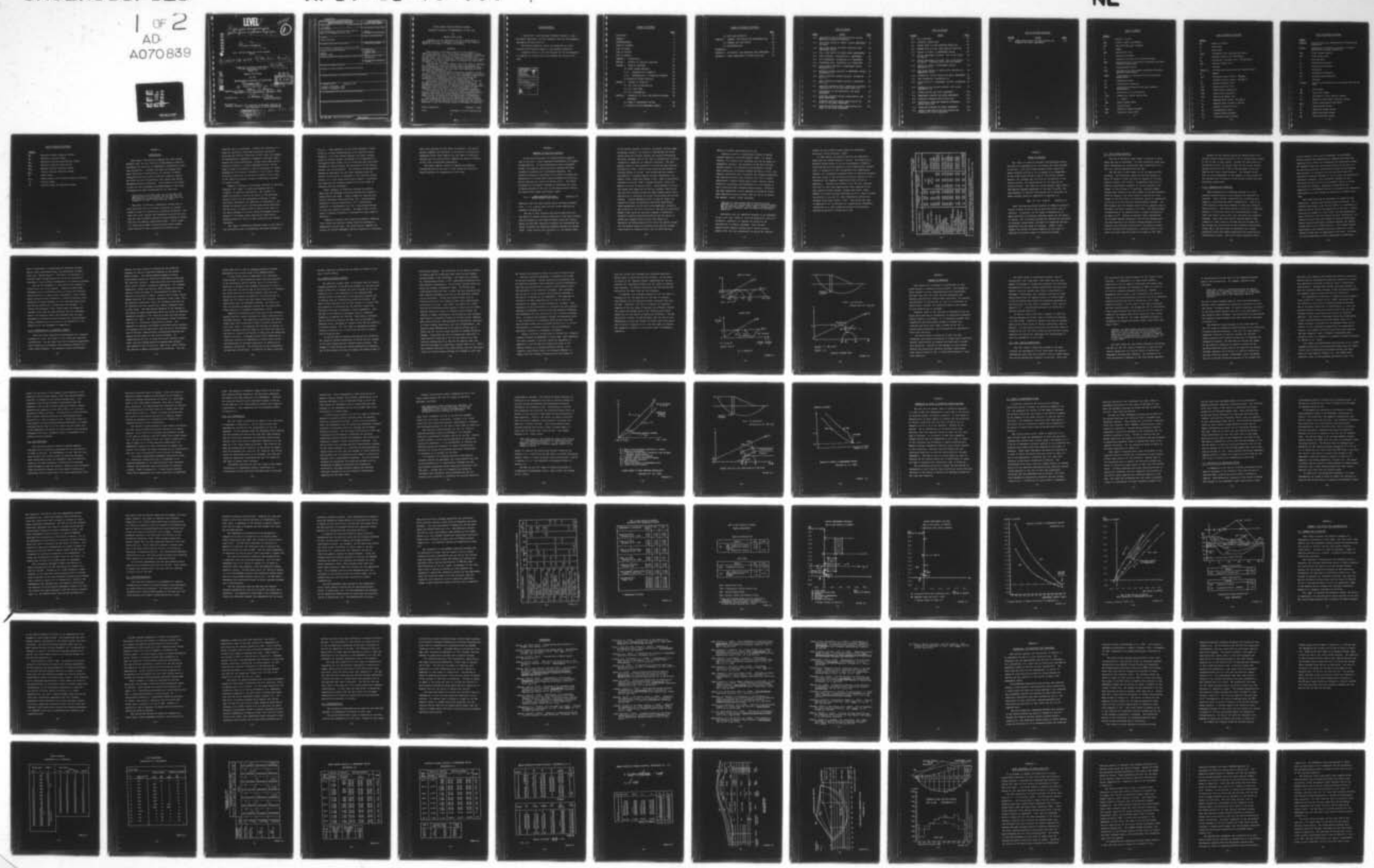

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TOTAL STRESS VERSUS EFFECTIVE STRESS

STABILITY ANALYSIS OF EMBANKMENTS ON SOFT CLAY

by

STEVEN CRAIG BOYCE

Submitted to the Department of Civil Engineering on December 18, 1978, in partial fulfillment of the requirements for the degree of Master of Science.

ABSTRACT

This thesis investigates the use of two types of stability analysis, i.e. total stress and effective stress, for evaluating the factor of safety of embankments constructed on soft clay. Three classes of stability problems are defined based upon the drainage conditions during construction and shear. The use of both types of analyses for each class of stability is discussed.

The factors of safety computed for both types of analyses are compared for three situations: eight case studies of actual embankment failures; variation with embankment height for seven embankments; and the long term stability for two hypothetical embankments.

Two cases received special attention: variation in the factor of safety with height for a test embankment constructed to failure in Portsmouth, N.H. and the long term stability of a hypothetical embankment resting on a soil having the anisotropic strength properties of Boston Blue Clay.

For design of embankments and determination of the end of construction stability the total stress analysis is recommended as being more reliable and easier to apply. The undrained strength should be determined by applying Bjerrum's empirical correction factor to field vane tests or from normalized soil properties such as the SHANSEP technique.

For variation of the factor of safety with embankment height, either the total or effective stress analysis may be used but the effective stress analysis will always give a higher factor of safety for the same height of embankment.

For long term stability the effective stress analysis is easy to apply and will give results significantly higher than a total stress analysis but in either case the long term situation is not the critical class of embankment stability.

Thesis Supervisor: Charles C. Ladd

Title: Professor of Civil Engineering

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LIST OF SYMBOLS

SYMBOL	
a	$\frac{1}{2}(S_u(V) + S_u(H))$
ADP	Active, Direct and Passive
b/a	$(\tau_{ff}(45^o))/(\tau_{ff}(H) \times \tau_{ff}(V))$
$\bar{B}$	$\Delta u/\Delta\sigma_v$
BBC	Boston Blue Clay
C	Undrained strength
$\bar{c}$	Cohesion intercept for effective stress
$\overline{CAU}$	Anisotropically Consolidated Undrained with pore pressure measurements
CD	Consolidated Drained
$\overline{CIU}$	Isotropically Consolidated Undrained with pore pressure measurements
$\overline{CK_oU}$	Consolidated K_o Undrained with pore pressure measurements
CR	Compression Ratio
CU	Consolidated Undrained
$\overline{CU}$	Consolidated Undrained with pore pressure measurements
c_v	Coefficient of consolidation
CVVC	Connecticut Valley Varved Clay
d	$1 - (b/a)^2$
DSS	Direct Simple Shear
e	$(1-K_s)/(1+K_s)$
ESA	Effective Stress Analysis
ESP	Effective Stress Path

LIST OF SYMBOLS CONTINUED

SYMBOL	
FS	Factor of Safety
FV	Field Vane
H	Horizontal
ICES	Integrated Civil Engineering System
K_o	Lateral stress ratio $\bar{\sigma}(H)/\bar{\sigma}(V)$
K_s	Anisotropic strength ratio $\tau_{ff}(H)/\tau_{ff}(V)$
KSF	Kips per Square Foot
L	Loading
LEASE II	Limiting Equilibrium And Soil Engineering II
M	Moment
OCR	OverConsolidation Ratio $\bar{\sigma}_{vm}/\bar{\sigma}_{vo}$
p	Average deviator stress $(\sigma_1+\sigma_3)/2$
$\bar{P}_c$	Preconsolidation load
PCF	Pounds per Cubic Foot
PSC	Plane Strain Compression
PSE	Plane Strain Extension
PSF	Pounds per Square Foot
q	Maximum shear stress $(\sigma_1-\sigma_3)/2$
q_f	Maximum shear stress at failure
R	Radius of failure circle
RR	Recompression Ratio
S_d	Drained shear strength
S_u	Undrained shear strength

LIST OF SYMBOLS CONTINUED

SYMBOL	
SHANSEP	Stress History And Normalized Soil Engineering Properties
STAB3D	Program for 3D stability analysis, includes anisotropic strength capability
TSA	Total Stress Analysis
TSP	Total Stress Path
u	Pore pressure
u_e	Excess pore pressure
u_o	Initial pore pressure
U	Unloading
UC	Unconfined Compression
UU	Unconsolidated-Undrained
V	Vertical
YLIGHT	Yield Locus Influenced by Geological History and Time
γ	Unit weight
γ_t	Total unit weight
θ	Inclination of the failure surface
μ	Bjerrum's field vane correction factor
ρ_{cf}	Final consolidation settlement
σ	Total normal stress
$\bar{\sigma}$	Effective normal stress
σ_1	Major principal stress
σ_3	Minor principal stress

LIST OF SYMBOLS CONTINUED

SYMBOL	
$\bar{\sigma}_f$	Effective normal stress at failure
$\bar{\sigma}_v$	Effective vertical stress
$\bar{\sigma}_{vc}$	Effective vertical consolidation stress
$\bar{\sigma}_{vf}$	Effective vertical final stress
$\bar{\sigma}_{vm}$	Maximum vertical effective stress
$\bar{\sigma}_{vo}$	Initial vertical effective stress
τ	Shear stress
τ_{ff}	Shear stress on the failure surface at failure
ϕ	Friction angle
$\bar{\phi}$	Friction angle for effective stress

CHAPTER 1

INTRODUCTION

This thesis discusses and compares the total stress analysis (TSA) and the effective stress analysis (ESA) for estimating the stability of embankments constructed on soft clay foundations. Although both types of analysis have their theoretical justification, the factors of safety calculated from each produce different numerical results and their use and interpretation have caused much debate amoung geotechnical engineers. This research is an effort to sort out the advantages and disadvantages of each type of analysis and make recommendations for their use.

> "If we are wise we will learn all we can about the implications of both methods (TSA and ESA) and be in a position to apply whichever one best suits the situation at hand." (Peck and Lowe, 1960)

The study is limited to embankments constructed on soft clays and does not include excavations or natural slopes in clays or construction on granular materials. The term "soft clay" implies that embankment construction will load the clay foundation beyond its preconsolidation stress and into the normally consolidated range. "Stiff clays", i.e. those which remain overconsolidated after loading would usually not have a problem with stability and

therefore are not considered. Despite this limitation in scope the profession frequently encounters situations involving embankments constructed on soft clay, for example in transportation embankments (highways, railroads, rapid transit and bridge approaches), levees, earth dams and for material stockpiles. Because embankments often involve large volumes of earthwork, it may be economical to design embankments with factors of safety of 1.3 or lower. When using such relatively low factors of safety, the accuracy of the stability analysis becomes increasingly important for proper design.

Chapters 2 through 4 will discuss 'methods' of stability analysis, the 'types' of stability analysis and the 'classes' of stability repectively. The 'method' of stability analysis involves the choice of the potential failure surface and the assumptions necessary to make statically determinate the equations of equilibrium. All methods of analysis use a limiting equilibrium definition for the factor of safety which is equal to the ratio of the shear strength of the soil along the assumed failure surface to the shear stress required for equilibrium. Possible sources of error in this calculation are briefly discussed.

The 'type' of stability analysis refers to the form of the calculation used in determining the shear strength of

the soil. This expression of the shear strength is based either on a total stress analysis or an effective stress analysis. In a total stress analysis the shear strength along the failure surface is assumed to be independent of the applied total stress, but of course may vary with depth (changes in stress history) and inclination of the failure surface (due to anisotropy). The effective stress analysis assumes an expression for the shear strength along the failure surface which is dependent on the effective normal stress, i.e. total stress minus pore pressure. Procedures for measuring the strength parameters for both types of analyses are also discussed.

Three 'classes' of stability problems are defined based upon the drainage conditions assumed with respect to consolidation and shear. These classes are labeled after similar laboratory test conditions: "UU", unconsolidated undrained or end-of-construction; "CU", consolidated undrained or intermediate; and "CD", consolidated drained or long term stability analysis. The application of both types of stability analysis is discussed for each class of stability.

Chapter 5 contains case studies and examples comparing total stress and effective stress stability analyses of embankments on soft clay. The first section compares the results of several embankment failures reported in literature

which were analyzed by both types of analysis. The second section compares the variation in the factor of safety with embankment height when computed by both types of stability analyses and the third section compares the factor of safety of two fully consolidated embankments.

The final chapter contains a summary and presents recommendations for the use of total stress and effective stress analyses for embankments on soft clay.

CHAPTER 2

METHODS OF STABILITY ANALYSIS

In any earth structure or foundation which applies load to the soil, a most important consideration for design is deformations. To keep deformations within acceptable bounds, two separate calculations are usually performed: a stability analysis and a settlement analysis. The stability analysis is performed in an effort to assess the possibility of a gross shear failure beneath the structure and is reported as a factor of safety (F.S.). The factor of safety for foundation loadings is defined as the ratio of shear strength of the soil to the shear stress required for equilibrium (Lambe and Whitman, 1969).

$$F.S. = \frac{\text{Shear strength of soil}}{\text{Shear stress @ equilibrium}} \qquad \text{Equation 2-1}$$

Therefore when the shear stress equals the shear strength (F.S. = 1.0), a gross shear failure occurs resulting in excessive deformations.

The use of this limiting equilibrium definition for the factor of safety requires prediction of the most critical failure surface and the shear strength along this surface. There are more than twenty-five methods for determining the factor of safety for stability analysis (Whitman and Bailey, 1967). These methods vary according to: the assumed shape

of the failure surface (circular, log spiral, sliding wedge or general surface); the method of determining the normal stress along this failure surface and thereby determining the shear strength; and by the type of calculations involved (arthimetic, graphical or computer solutions). The most popular method is the procedure of slices as first próposed by Fellenius (1936), which is also known as the Swedish or Ordinary Method of Slices. This method has been adapted and modified to varying degrees by many researchers, e.g. Bishop (1955), Simplified Bishop (1955), Janbu's Generalized Procedure of Slices (1957), Morgenstern-Price (1965) and Spencer's method of slices (1967). These methods of slices all assume some shape for the failure surface, then divide the soil into vertical slices and apply moment and/or force equilibrium to calculate the normal forces along the failure surface. Various assumptions must be made to reduce the unknowns and make the equations statically determinate. These assumptions involve side force magnitude, direction and location of its line of thrust on the side of each slice. Wright (1969) gives an excellent summary of twenty-one different limiting equilibrium methods for the factor of safety and compares their results (See Table 2-1). Generally the methods agree within about ten percent except for the Swedish method in granular soils and the sliding block method in cohesive soils, both of which may give

factors of safety significantly too low.

Sources of error in calculating a factor of safety include (Barboteu, 1972 and Gilbert, 1974): the random testing of a variable soil, modeling the soil as layers of homogeneous material, the selection of the most critical failure surface, the effects of end restraint in actual three dimensional situations (Baligh and Azzouz, 1975), and testing procedures used to evaluate the strength parameters. The testing procedures may introduce error due to sample disturbance, triaxial versus plane strain testing, strain rates, soil anisotropy and determination of the changes of the in situ principle stresses during loading. Therefore, the input parameters of soil strength and geometry become more important sources of error than the choice of method. For example, Johnson (1974) concludes:

> "The use of total stress versus effective stress analyses and the various ways in which design strengths can be selected produce a wide range of safety factors and are more important than the method for analyzing stability."

The method used for stability analysis of an embankment is up to the user, based on one's own experience; choice of arithmetic, graphical or computer solutions and the availability of computer programs. Even the most sophisticated computer program cannot insure accuracy greater than the input parameters nor should the computer

results be used without being checked for reasonable solutions (Little and Price, 1958).

In this thesis the methods used are the Simplified Bishop and the STAB3D methods of slices. The Simplified or Modified Bishop method of slices as programed on LEASE II (Limiting Equilibrium Analysis in Soils Engineering) (Dawson, 1972), a subsystem of ICES (Integrated Civil Engineering System), is a reasonably accurate method of stability analysis and well suited to the circular arc failure analyzed in the Portsmouth, New Hampshire Experimental Test Section in Appendix A. Although this method is not the most rigorous, its error is less than eight percent when compared to any more rigorous method (Wright, Kulhawy and Duncan, 1969). STAB3D is a method recently developed at M.I.T. which includes the ability to account for three dimensional effects and strength anisotropy (Azzouz, 1977). This method was used in Appendix B for the analysis of a hypothetical embankment constructed on a clay foundation having the anisotropic strength properties of Boston Blue Clay.

EQUILIBRIUM CONDITIONS SATISFIED BY VARIOUS METHODS OF STABILITY ANALYSIS
(Wright 1969)

METHOD OF ANALYSIS	EQUILIBRIUM CONDITIONS SATISFIED OVERALL			EACH SLICE			SLIP SURFACE
	M	V	H	M	V	H	
(Ø=0) Method	Yes	Yes*	Yes*	Not a Method of Slices			Circular Arc
Log Spiral	Yes	Yes*	Yes*				Log Spiral
Plane Shear Surface, Culmann	Yes	Yes	Yes				Plane
Friction Circle Method	Yes	Yes	Yes				Circular Arc
Frohlich	Yes	Yes	Yes				Circular Arc
Bell	Yes	Yes	Yes				General Shape
Swedish, Fellenius	Yes	No	No	No	No	No	Circular Arc
Petterson	Yes*	Yes*	Yes*	Yes	Yes	Yes	General Shape(1)
Fellenius Rigorous Graphical	Yes*	Yes*	Yes*	Yes	Yes	Yes	General Shape(1)
Raedachelders	Yes*	Yes*	Yes*	Yes	Yes	Yes	General Shape(1)
Modified Bishop	Yes	Yes*	No	No	Yes	No	Circular Arc
Bishop Rigorous	Yes	Yes*	Yes*	Yes	Yes	Yes	Circular Arc
Monveiller	Yes	Yes*	Yes*	Yes*	Yes	Yes	General Shape
Spencer	Yes	Yes*	Yes*	Yes	Yes	Yes	General Shape(1)
Morgenstern & Price	Yes*	Yes*	Yes*	Yes	Yes	Yes	General Shape
Janbu et. al. Horizontal Side Forces	No	Yes*	Yes*	No	Yes	Yes	General Shape
Lowe & Karafiath	No	Yes*	Yes*	No	Yes	Yes	General Shape
Modified Swedish Method	No	Yes*	Yes*	No	Yes	Yes	General Shape
Janbu Generalized Procedure of Slices	Yes*	Yes*	Yes*	Yes	Yes	Yes	General Shape
Seed & Sultan	No	Yes*	Yes*	No	Yes	Yes	2 Sliding Wedges
Sliding Block	No	Yes*	Yes*	No	Yes	Yes	3 Sliding Blocks

* This condition of equilibrium is implicitly satisfied as a result of the direct consideration of other equilibrium conditions.
1 The original presentation of this method was for a circular arc only.

TABLE 2-1

CHAPTER 3

TYPES OF ANALYSES

The 'type' of stability analysis distinguishes between the choice of expressions for calculating the shear strength of the soil. The total stress analysis (TSA) inputs the shear strength directly as it is assumed to be independent of the total normal stress acting on the failure surface. However, the strength may vary with the inclination of the failure surface, the stress history of the clay and the level of strain. The effective stress analysis (ESA) inputs the drained strength parameters of the clay ($\bar{c}$ and $\bar{\phi}$), the pore pressure (u) and the total stress (σ) to calculate the shear strength using the expression in Equation 3.1.

$$\tau_{ff} = \bar{c} + (\sigma - u)\tan\bar{\phi} \qquad \text{Equation 3.1}$$

Most limiting equilibrium methods of stability analysis and all methods of slices may be used with either type of analysis. In general, the two types of analyses do not agree and result in different numerical values for the factor of safety. The following sections will discuss the definition of strength, stress paths, and means for measuring the soil parameters for each types of analysis. Chapter 5 will compare the factors of safety computed by both types of analyses for several classes of stability problems.

3.1 TOTAL STRESS ANALYSIS

The use of the TSA in this thesis is limited to cases where the clay is saturated. For soft foundation clays upon which embankments are constructed, this is generally true and not a serious limitation for the TSA.

The TSA used in this thesis is not the same as a ϕ=0 analysis. The theoretical basis for the ϕ=0 analysis is given by Skempton (1948) and shown in Figure 3-1 for some typical triaxial test results. Recognizing that water is nearly incompressable compared to the soil skeleton, any change in total stress on a saturated, undrained sample during shear results in an equal change in the pore pressure. Since an change in the total stress is carried by a like change in the pore fluid, no additional stress is transferred to the soil and the effective stress remains constant. The ϕ=0 analysis uses the maximum shear from an undrained test to determine the shear strength of the soil. The failure plane corresponding to the maximum shear is always orientated at an angle of 45°. Bishop and Bjerrum (1960) demonstrated for a simple example excavation that the ϕ=0 analysis gave the same results as an effective stress analysis for the stability but different locations for the failure surface. The ϕ=0 analysis will always give a failure surface at 45° while the actual failure occurs at an angle of $45^{\circ} + \bar{\phi}/2$.

Because the ϕ=0 analysis gives the maximum shear and a failure surface always inclined at 45° to the principal stress it is not used in this thesis. Instead the shear stress on the failure surface at failure (τ_{ff}) and the actual failure surface inclined at $45^{\circ} + \bar{\phi}/2$ to the principal stress are used for all total stress analyses. The strength of the soil is still independent of the changes in total stress but the strength and inclination of the failure surface are more accurately represented.

3.1-1 UNDRAINED SOIL BEHAVIOR

When determining the undrained strength of a soil several characteristics of soil behavior are important to consider. These include: stress history, normalized soil behavior, anisotropy and strain compatability. Ladd et al. (1977) present an excellent summary of these soil behavior characteristics. The stress history of the soil is important because of the effects which the overconsolidation ratio (OCR) and lateral stress ratio (K_o) have upon the undrained strength. Many clay soils exhibit similar behavior when compared to other tests on the same soil at the same OCR. By dividing the test results by the effective consolidation stress ($\bar{\sigma}_{vc}$) they may often be represented by a single normalized curve which makes presenting and evaluating the clay behavior easier. The use of these normalized curves

of soil behavior led to the developement of the SHANSEP (Stress History And Normalized Soil Engineering Properties) (Ladd and Foott, 1974) for determining the undrained strength. The SHANSEP method uses the stress history of the soil i.e. the existing effective consolidation stress and the maximum past pressure, to determine the OCR and from the plot of normalized undrained strength vs OCR to select the appropriate normalized strength. This normalized strength is then multiplied by the effective consolidation stress to obtain the undrained strength. Appendix B demonstrates the application of the SHANSEP method for a hypothetical embankment constructed on a soil having the properties of Boston Blue Clay.

Soil anisotropy may be considered to consist of two components: inherent and stress induced anisotropy. The inherent anisotropy is due to soil fabric or orientation of particles which result from deposition of the soil. Varved clays are an example of a soil which even macroscopically have variation in the soil properties with orientation, but almost all soils have some degree of inherent anisotropy. Stress induced anisotropy is present whenever the lateral stress ratio (K_o) is other than one. This stress induced anisotropy is not measured correctly unless the in situ lateral stress ratio is duplicated in the test. Of course it is the combination of both components of anisotropy

which is important in determining the undrained strength and is often represented by K_s, the anisotropic strength ratio ($K_s = \tau_{ff}(H)/\tau_{ff}(V)$). This ratio of horizontal to vertical shear strength may be as low as 0.3 and must be considered in any accurate determination of undrained strength.

Just as the strength of a clay varies with changes in failure mode, so do the strains. Because the peak shear strengths occur at different levels of strain for different failure modes (i.e. plane strain compression, direct simple shear and plane strain extension) a stability analysis which considers the peak strengths to occur simultaneously along the failure surface will overestimate the strength. To correct the strengths, their values might be averaged at constant strain and the peak strength from this averaging used in the analysis rather than assuming that the peak strengths for different failure modes occur simultaneously. Strain compatibility is included in the evaluation of the stability for the embankment in Appendix B.

3.1-2 DETERMINATION OF UNDRAINED STRENGTH

There are three techniques for determining the undrained strength of a soil: lab tests, in situ tests and empirical correlations. For lab tests there are two general approaches used in practice to determine the undrained strength for a total stress analysis. One assumes a unique relationship

between the water content at failure and the undrained strength so that all tests are conducted at the natural water content. The second approach assumes a unique relationship between the effective consolidation stress and the undrained strength. Tests which are run at the natural water content include: unconfined compression, "UU" triaxial compression (Bishop and Bjerrum, 1960) Labvane and Torvane. Tests in which the sample is reconsolidated to match the effective consolidation stress include: direct shear, direct simple shear and "CU" triaxial compression tests (Ladd, 1971). By conducting laboratory tests, the boundary conditions can be controlled and the results more easily interpreted than in situ tests. The most serious drawback to lab testing is the effect of sample disturbance which can reduce the measured undrained strength in "UU" tests by as much as fifty percent (Ladd et al. 1977). The "UU" test may give correct solutions only due to a fortuitous cancelation of errors. The reduction in strength from sample disturbance and an increase in strength from the rapid shear rate and use of the strength due to vertical loading without considering anisotropy may cancel out and result in the correct strength. By consolidating the soil to match the in situ lateral stress ratio (K_o) and loading beyond the maximum past pressure with rebound to the *in situ* OCR, Ladd et al. (1977) believe the effects of sample disturbance are minimized. The test

results may then be used by applying normalized strength properties such as those used in the SHANSEP method.

In situ tests offer an improvement over laboratory tests because of their generally lower costs and reduction in the disturbance before testing, but it is often more difficult to determine the boundary conditions for thé in situ tests and the results may be more difficult to interpret. The field vane test is the most popular field test for measuring the undrained strength but its failure mode is unique and its strain rate much higher than would be experienced under an embankment loading. The Dutch Cone test is becoming increasingly popular but its results are often difficult to interpret (Schmertmann, 1975).

Empirical correlations are often used in soil mechanics because practice usually preceeds the theory required for a thorough analysis. Most empirical correlations are based on index tests such as Bjerrum's (1972) correction factor for field vane tests. By analyzing the circular arc failure of sixteen embankment failures based on the field vane test, he was able to correlate the factor of safety and the Plasticity Index of the soil. Although Bjerrum (1973) explained the correction factor by a combination of strain rate effects and anisotropy for different plasticity it remains as an empirical correlation. The Dutch Cone is also interpreted by empirical correlations. In addition to these correction

factors, empirical correlations are useful as checks on any lab or field testing.

3.2 EFFECTIVE STRESS ANALYSIS

The effective stress 'type' of analysis uses the drained strength parameters ($\bar{c}$ and $\bar{\phi}$), pore pressure (u) and the total normal stress (σ) to express the shear strength of the soil (Equation 3.1). This expression is the classical Mohr Coulomb failure criteria and is the generally accepted way to analyze most soil engineering stability problems. The ESA is often preferred over the TSA because it is known that soil behavior is controlled by the effective stress and therefore this type of analysis is more fundamental. Also the ESA is not limited to saturated soils or undrained loading as is assumed for the TSA. Many kinds of problems can be correctly solved by the ESA, such as problems in granular soils, unloading in cohesive soils and the stability of natural slopes. The "CD" class of embankment problems (defined in Section 4.2) on soft clay can also be easily solved using the ESA.

To use the ESA the failure envelope must be defined by the effective strength parameters ($\bar{c}$ and $\bar{\phi}$) and the changes in the total stress and pore pressure must be known. The changes in total stress are usually determined by applying one of the methods of slices to compute the stress normal to

the failure surface. The evaluation of the failure envelope is usually made by conducting shear tests on soil samples reconsolidated in the laboratory. The tests can be either CD, consolidated-drained, or $\overline{CU}$, consolidated-undrained with pore pressure measurements. The CD tests include direct box shear and triaxial compression tests. The $\overline{CU}$ test aré consolidated-undrained test in which the pore pressures have been measured throughout the test so the effective stresses can be calculated. Bishop and Bjerrum (1960) give an excellent review of all triaxial tests, their applications and sources of error. Ladd (1971) and Ladd et al. (1977) discuss the difficulties and sources of error for all tests. Possible sources of error in triaxial tests include: the effects of sample disturbance; end restraints; pore pressure equilization; correction factors for filter strips, membrane and piston friction; back pressure for saturation; efficency of filter strips; anisotropy; and test interpretation. The friction angle ($\overline{\phi}$)measured at the maximum stress (q_f) is lower than the friction angle measured at maximum obliquity (σ_1/σ_3). Anisotropy may also be significant especially in lean sensitive clays where the friction angle at q_f from vertical loading may be as much as ten degrees lower than the friction angle for horizontal loading (Ladd et al. 1977). The cohesion intercept ($\overline{c}$)is often smaller than the correction factor for filter strips although for loadings on soft clay

the cohesion can safely be taken to be zero for most soils.

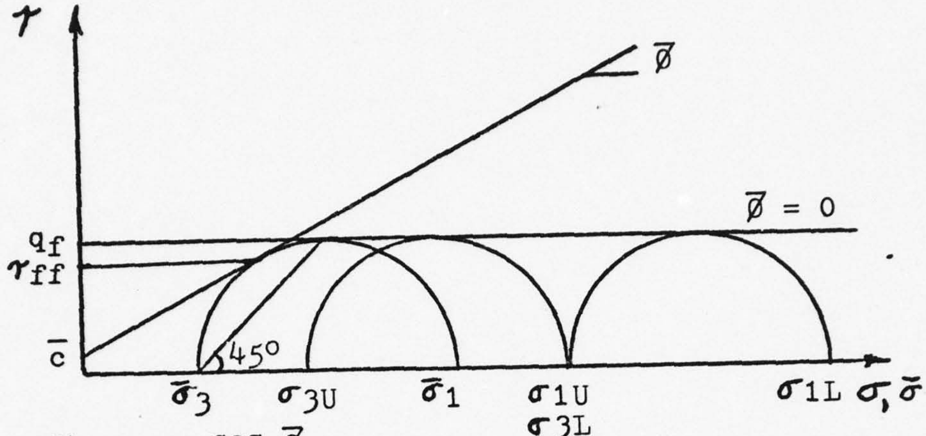
The most difficult parameter to determine for an ESA is the pore pressure. During construction piezometers can be installed throughout the foundation clay and measurements from the piezometers used to create a pore pressure field for evaluation of the effective stresses. This technique can be used to relate directly the piezometer readings to the factor of safety for the embankment. This approach is attractive for many engineers but it does not give any indication of changes in pore pressure due to shear. This technique of using piezometers to determine the pore pressure beneath the embankment does not help the engineer designing the embankment and only serves as a tool to monitor construction. To predict pore pressure beneath the embankment prior to any construction is extremely difficult and even with pore pressure models such as YLIGHT (Section 4.1) it is not yet possible to accurately predict the pore pressures generated throughout an embankment foundation.

Figure 3-2 shows a stress path to failure for a "typical element" beneath an embankment on soft clay. When computing the shear strength of the soil using the expression in Equation 3.1 for the ESA one obtains the shear on the failure surface at a constant effective stress. For normally consolidated clays, this strength, τ_{ff}ESA, is higher than the strength obtained from a TSA because it

does not include the increased pore pressures generated during shear to reach the failure envelope. As the state of stress on the "typical element" approaches the failure envelope the differences between the two types of analysis will decrease. At failure conditions both the ESA and TSA should give the same value for the strength.

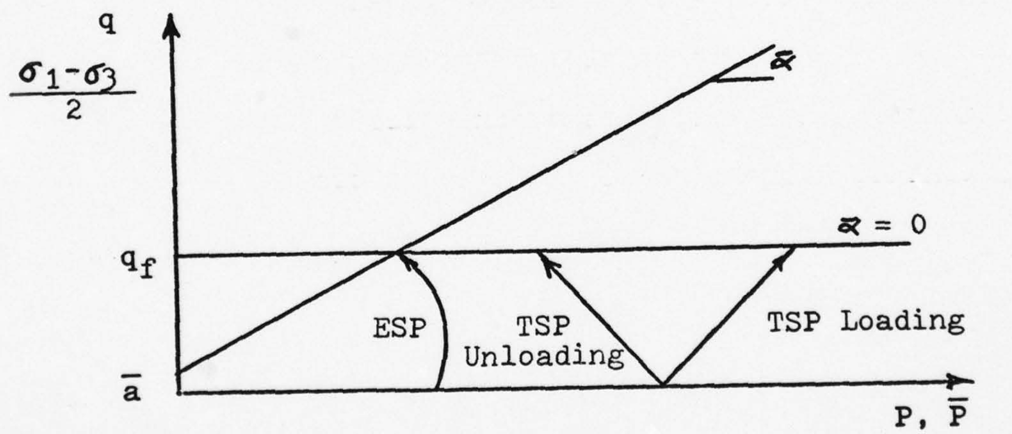
In general, the strengths computed by the two types of analyses are not the same and for the case of normally consolidated clay the ESA gives higher strengths than does the TSA. Therefore, the factor of safety for stability computed by the two types of analyses are different because the values of strength used in the limiting equilibrium definition for the factor of safety are different. The ESA should give a factor of safety higher than the TSA for all embankments on soft clay except at failure when both should give a factor of safety of 1.0. The two types of analyses are compared in Chapter 5 for several cases of embankment stability.

MOHR'S CIRCLE



$\tau_{ff} = q_f \cos \bar{\phi}$

STRESS PATH



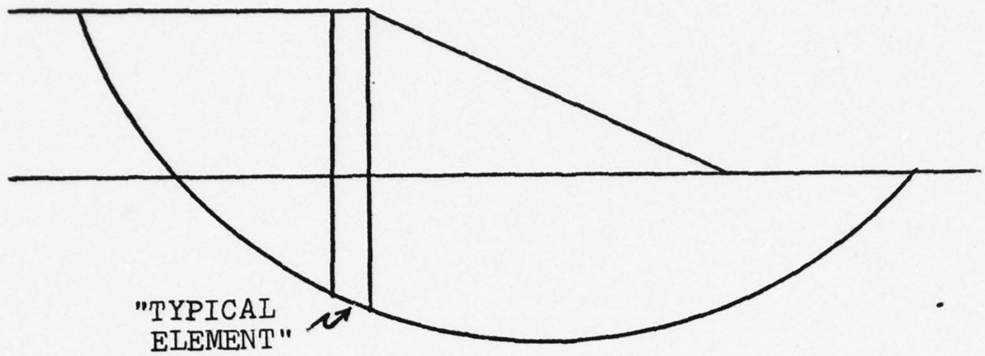
$\frac{\sigma_1 + \sigma_3}{2}$ $\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}$

$\bar{a} = \bar{c} \cos \bar{\phi}$

$\tan \bar{\alpha} = \sin \bar{\phi}$

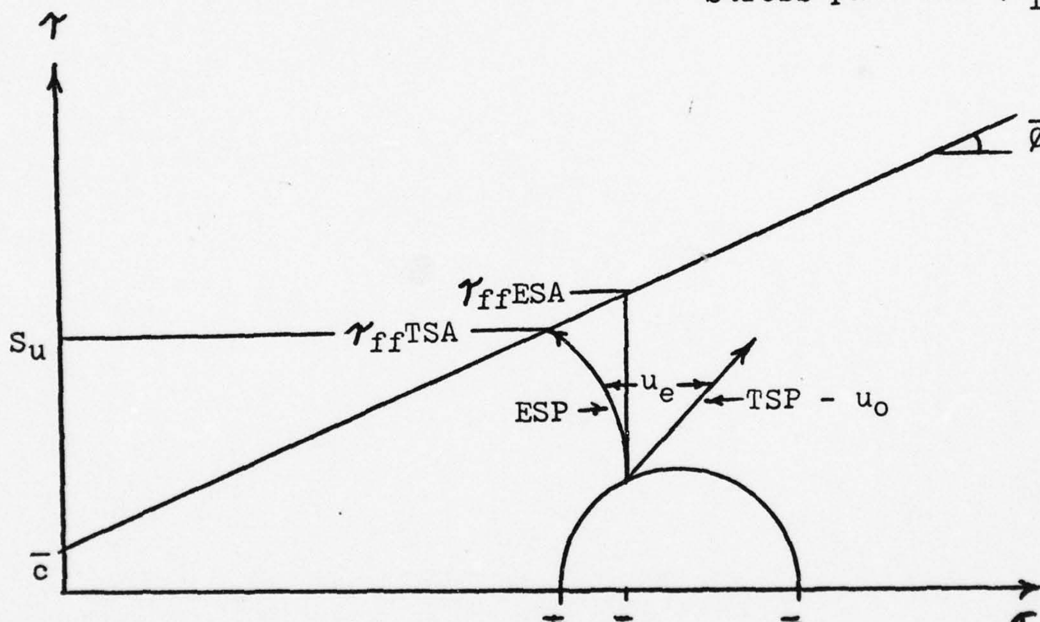
$\phi = 0$ ANALYSIS

FIGURE 3-1



NOTE - u_o neglected

- Stress path for (τ_{ff}, σ_f)



$\tau_{ff}ESA = \bar{c} + \bar{\sigma}_f \text{ TAN } \bar{\phi}$

$\tau_{ff}TSA = S_u$

"TYPICAL" STRESS PATH

FIGURE 3-2

CHAPTER 4

CLASSES OF STABILITY

Any construction (loadings or excavations) on clay soils results in changes in pore pressure. This change is often assumed to occur instantaneously with construction and then vary with time until the pore pressures have reached equilibrium or steady state conditions. To evaluate the stability of construction on clay it is convenient to define three 'classes' for stability based on the changes in pore pressure due to drainage.

The first class is the case when no drainage occurs and hence there has been no dissipation of excess pore pressures during loading or unloading. It is frequently assumed that construction proceeds rapidly enough to satisfy this condition and hence this class is known as the "UU" = unconsolidated-undrained, end-of-construction, immediate or short term stability.

The second class of stability is when the clay foundation has reached equilibrium or steady state conditions wherein consolidation is complete and there are no excess pore pressures. Failure is then assumed to occur slowly without any excess pore pressure developed during shear. This class is known as "CD" = consolidated-drained or long term stability.

The third class is intermediate between 'end-of-construction' and 'long term', when there has been some degree of drainage so excess pore pressures have partially dissipated but not yet reached equilibrium conditions and failure occurs so rapidly that there is essentially no drainage. Alternately, the foundation clay may be fully consolidated under the applied loads, but failure again occurs so rapidly that excess pore pressures generated by the failure do not have time to dissipate. This class is known as "CU" = consolidated-undrained, intermediate or partial drainage stability.

In theory, each of the three 'classes' of stability could be analyzed by either 'type' of stability analyses, i.e. TSA or ESA. However in practice one type of analysis may be much better suited to handle a particular class of problem. The following sections will review each class of stability by both the total stress and the effective stress analyses and discuss the advantages and disadvantages of each for embankments on soft clay.

4.1 "UU" END-OF-CONSTRUCTION

The "UU" class of stability problems is the most critical for embankments on soft clays where the applied stresses are such that the foundation clay is loaded beyond the in situ maximum past pressure of the soil. (Note that

for excavations and natural slopes the "CD" class is most critical). An embankment on a deep truely normally consolidated clay is usually constructed much faster than excess pore pressures can dissipate so there is little or no error in assuming undrained conditions. However, embankments built slowly, over a relatively thin layer of clay or on an overconsolidated clay would not be an undrained loading condition and must be treated as a "CU" class stability problem. In any case, if the clay is saturated and then loaded quickly, the assumptions for a conventional total stress analysis have been met and the "UU" class of stability has usually been solved using the total stress analysis. The TSA for "UU" class embankment problems is recommended by Bishop and Bjerrum (1960), Ladd (1971) and Janbu (1977).

> "However, the main reason for using a total stress approach for short-term undrained conditions in clay is to avoid predicting the pore pressures and hence the effective stresses. This may at times be justified for embankments and footings when the load on the clay exceeds the preconsolidation load." (Janbu, 1977)

The only reason for any further discussion concerning the TSA for the "UU" class of stability is that the TSA doesn't always give the correct factor of safety and embankment failures have resulted. The problem is not with the method of analysis or the total stress approach

to the problem but in the value of the undrained strength used in the calculations. For example, Bjerrum (1972) concludes:

> "Obviously, there is nothing wrong with the general principle of the presently used methods of computing the stability. When they do not work, it is the shear strength introduced in the computation which is incorrect."

To correct input strength based on field vane measurements Bjerrum (1972) recommended an empirical correction for field vane tests. As stated above, it does not imply any correction to the total stress analysis but only to the measurement of the undrained shear strength. The difficulties of determining the undrained shear strength from other tests were discussed in Section 3.1.

The effective stress analysis is not often used for the "UU" class of stability problems because of the difficulty in predicting pore pressures. Also the "UU" class implies undrained conditions to failure, whereas the effective stress analysis uses a drained stress path to determine the strength at failure. The ESA does not include any change in the effective stress due to shear to failure as the existing effective stress used in the equation for failure strength (Equation 3.1) results in a failure condition at the same effective stress. Schmertmann (1975) recommends the use of the ESA exclusively because of the insight into

the basic soil behavior and because the empirical correction factors used in obtaining the undrained strength for the TSA e.g. Bjerrum's field vane correction, have no theoretical justification. Perhaps when testing technology advances so that the undrained shear strength is more accurately measured his argument against using a TSA will be eliminated.

For embankments constructed on lightly overconsolidated clays (OCR < 2.5), Leroueil et al. (1978) have shown that the foundation clay acts as a partially drained material, due to the high coefficient of consolidation c_v, until the preconsolidation stress is reached at which time the clay becomes normally consolidated and acts as an undrained material. The effects of drainage during the recompression of the overconsolidated clay would usually result in a slight increase in strength and the analysis based on the initial in situ undrained strength would be conservative. However, with highly structured, sensitive clays (such as the Lake Champlain clays in Canada) recompression may actually result in a decrease in undrained strength according to Tavenas et al. (1978).

For lighly overconsolidated clays Leroueil et al. (1978) recommend prediction of pore pressures based upon the YLIGHT (Yield Locus Influenced by Geological History and Time) model of clay behavior (Figure 4-1). The YLIGHT model predicts a rate of pore pressure generation of $\overline{B}_1 = 0.6$,

where $\overline{B}$ is the ratio of the change in pore pressure to the change in total vertical stress, until the preconsolidation load ($\overline{P}_c$) is reached at the critical height (H_{cr}). The foundation clay becomes normally consolidated under the embankment load at the critical height and further loading generates pore pressure at the rate of $\overline{B}_2 = 1.0$ in a true undrained loading condition. At conditions of local yielding with a strain softening material the pore pressures are generated at a rate in excess of one. Though this model is very helpful in depicting pore pressure generation in lightly overconsolidated clays it is only valid for the center line of the embankment and cannot be used to predict pore pressure throughout the entire foundation which is necessary for an effective stress analysis.

4.2 "CD" LONG TERM

The long term class of stability assumes complete drainage with no excess pore pressures due to construction or shear to failure. For embankments on soft clay the long term is never the critical class for stability analysis. With consolidation under the embankment load the effective stress in the clay increases, resulting in an increased shear strength so that the factor of safety is also increased. Often the factor of safety for this class of stability is calculated only to insure that it is above some preset value

arbitrarily selected prior to design. Since this class of stability assumes complete consolidation and no change in effective stresses during shear to failure i.e. they remain equal to the consolidation stresses, there are no excess pore pressures. The equilibrium pore pressures can therfore be determined from the water table or if steady state flow exists from a flow net. Figure 4-2 shows the stress path for a point on the failure surface of a "typical element" beneath an embankment: 0-1 represents the loading due to construction and 1-2 the consolidation.

The effective stress analysis is used almost exclusively for this class of stability problems because it is easily applied. Since there are no excess pore pressures the ESA is the same as assuming a drained loading from 2-ESA2, Figure 4-2. The use of the ESA for the "CD" class of stability is easy to apply but of little practical significance for embankments (except for those constructed in stages, see Section 4.3) because the higher strengths due to consolidation make the 'end-of-construction' more critical for design.

If a fully consolidated embankment was quickly loaded i.e. additional construction due to enlargement of an existing embankment or an extraordinarily heavy transient loading, the additional load would produce an undrained loading, 2-TSA2 in Figure 4-2. A total stress analysis would now be appropriate for these conditions of additional

load. The question is whether a rapid failure or the very slow drained failure assumed in the ESA better represents the actual long term stability of an embankment. Treating the long term stability with undrained loading to failure is the same as a "CU" class stability problem with complete consolidation. This comparison will be discussed further in Section 5.3.

4.3 "CU" INTERMEDIATE

The "CU" class of stability can apply to two conditions of embankment construction. One of these is when some drainage has occurred but the consolidation is not yet complete. If the drainage under the embankment were complete the case becomes the same as the long term class assuming that no excess pore pressures are developed during actual failure, and if the clay is completely undrained it is the same as a "UU" class stability problem. This would be the same as determining the variation in the factor of safety with the drainage time, with the "UU" class for the initial factor of safety when there is no drainage, the long term class for completely consolidated and the "CU" class for any time in between these extremes.

The second condition for the "CU" class is the staged construction of embankments wherein construction is interrupted to allow additional drainage before continuing

construction. This interruption in work could be due to seasonal weather changes, work strikes, modification to an existing embankment or an intentional delay to allow the clay beneath the embankment time to consolidate and gain strength so the embankment may be built higher than could be allowed in a single stage.

A total stress analysis for the "CU" class of stability implies an undrained failure starting from the existing effective consolidation stress. As the clay consolidates under the embankment load the excess pore pressures dissipate resulting in increased effective stresses and increased undrained strength. The two major difficulties in this type of analysis are estimating or measuring the existing effectives stresses beneath the embankment and relating the undrained strength to these consolidation stresses. One way to obtain the existing effective consolidation stresses in the clay foundation is to evaluate the preconsolidation stress history of the clay, the changes in total stress and pore pressure due to loading and the change in excess pore pressure due to consolidation drainage. Once the existing stresses are determined the undrained strength could be determined by applying normalized soil properties. Alternately, one might directly measure the existing undrained strength by in situ testing or undisturbed sampling and lab shear tests.

Turnbull and Hvorslev (1966) recommend the use of the total stress analysis for the "CU" class of stability problems, and state:

> "The appropriate shear strength is, therefore, the undrained strength of the soil after it has been consolidated to a condition corresponding to the estimated effective stresses in the field."

Ladd (1971) recommends the use of the TSA with SHANSEP method for this class of problems. Tavenas et al. (1978) used field vane tests after partial consolidation to measure the increased strength for the TSA in this class of stability.

The effective stress analysis for the "CU" class of stability problems requires knowledge of the existing effective consolidation stresses just as in the TSA. Failure is then assumed to occur without generation of additional excess pore pressures. It is inconsistent to consider the existing excess pore pressures to remain constant while allowing the excess pore pressures generated during shear to dissipate as is assumed for all ESA.

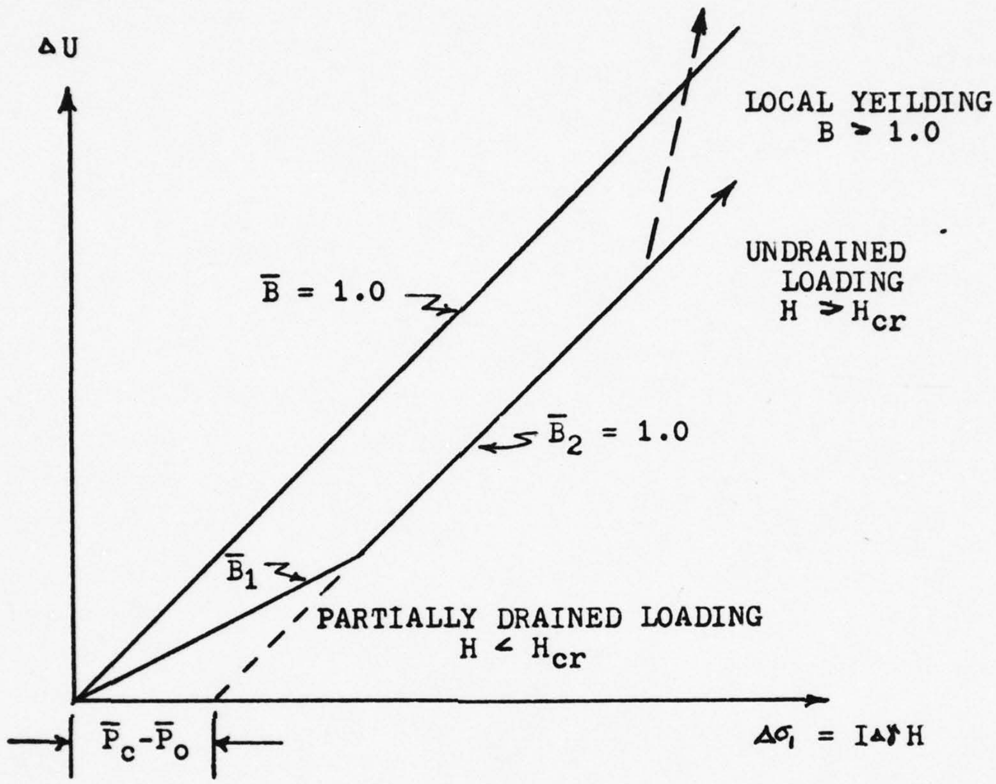
To determine the existing effective consolidation stresses beneath the embankment one could subtract the loss of pore pressure due to consolidation drainage from the pore pressure generated during loading as mentioned in the TSA above. Another way is to measure pore pressures throughout the foundation with piezometers and use the measured pore pressures to determine the existing effective

consolidation stresses. The effective stress analysis for stability of an embankment can then be directly related to the pore pressures measured from piezometers installed throughout the foundation clay. This ability to monitor pore pressures and calculate the effective stress factor of safety for any instant in time makes the ESA attractive to many engineers. However, the factor of safety calculated does not include changes in pore pressure resulting from shear due to actual failure. Since pore pressures may increase rapidly near failure loads, a larger margin of error must be allowed when using the ESA. For example Tavenas et al. (1978) state:

> "When some points of the foundation reach local failure . . . increased pore pressures are generated and the available strength decreases . . . as a result the factor of safety will decrease rapidly toward 1.0 at complete failure."

Figure 4-3 depicts the generalized behavior predicted by Tavenas et al. (1978) with the ESA factor of safety decreasing rapidly toward 1.0 after reaching local yielding near a factor of safety of 1.3. This behavior is discussed further in Section 5.2.

The ESA for the "CU" class of stability problems is recommended by Schmertmann (1975), Janbu (1977) and Tavenas et al. (1978).

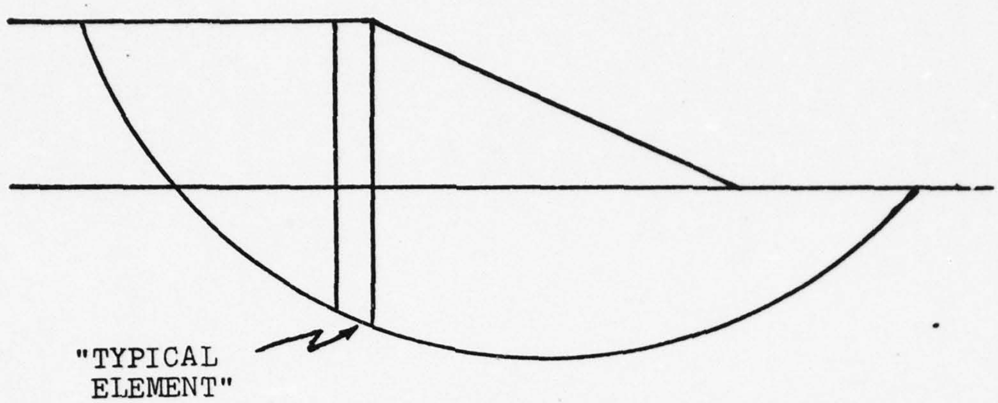


$\overline{B}$ = Rate of pore pressure generation = $\Delta U / \Delta\sigma_1$
H = Height of embankment.
H_{cr} = Critical height when foundation clay becomes normally consolidated.
$\underline{I}$ = Influence factor for loading.
$\overline{P}_o$ = Initial vertical effective stress.
P_c = Maximum past pressure.
U = Pore pressure.
γ = Total unit weight of embankment soil.
σ_1 = Total vertical stress.

YLIGHT MODEL OF PORE PRESSURE PREDICTION

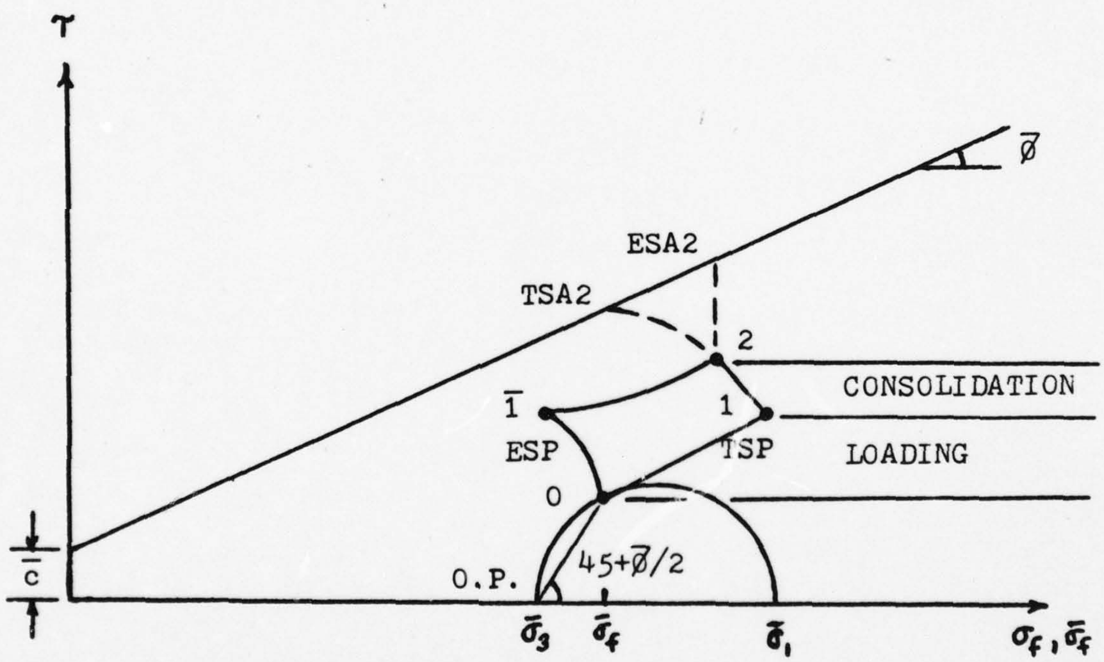
(Leroueil et al. 1978)

FIGURE 4-1



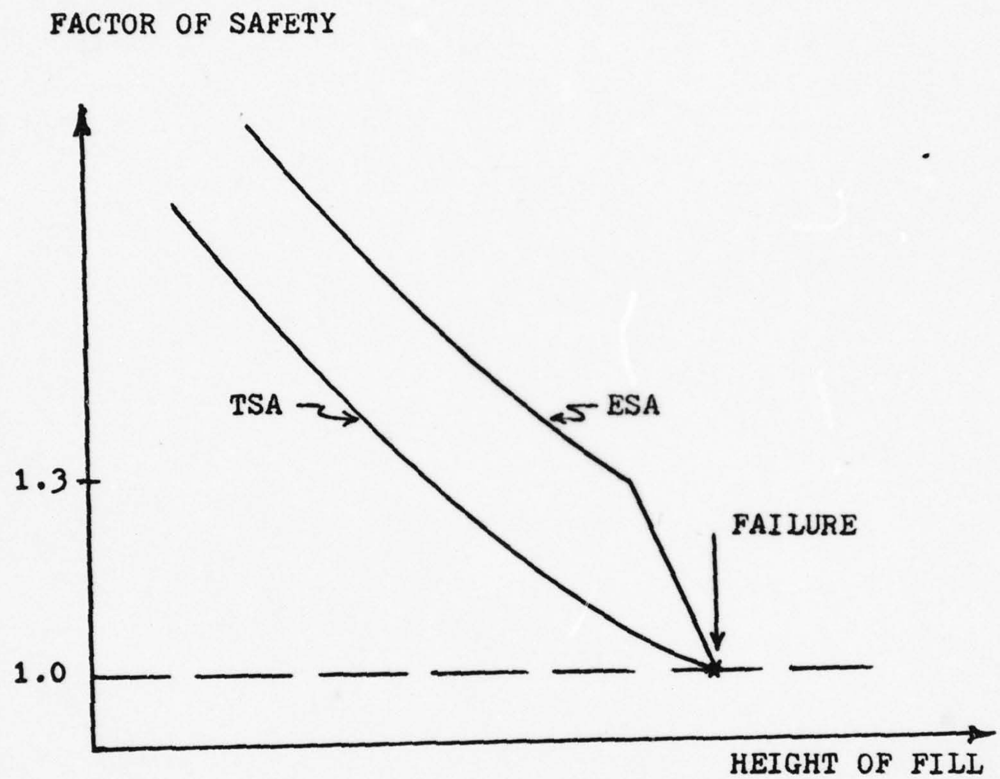
Note - U_o Neglected

- Stress Path for (τ_f, σ_f)



STRESS PATH FOR LONG TERM STABILITY ANALYSIS

FIGURE 4-2



FACTOR OF SAFETY vs EMBANKMENT HEIGHT

(Tavenas et al. 1978)

FIGURE 4-3

CHAPTER 5

COMPARISON OF TOTAL vs EFFECTIVE STRESS ANALYSES

The true test of either 'type' of stability analysis i.e. TSA or ESA, for embankments on soft clay is its ability to correctly predict a failure condition. If both types of analyses give an equal degree of accuracy for the prediction of failure conditions then the choice of which type of analysis to use is up to the engineer. However if one approach can be demonstrated to be significantly more accurate, then that approach should be used over the other type of analysis. In an effort to make this comparison between TSA and ESA for embankments on soft clay, a review of the literature was necessary to compile existing data. Despite numerous embankment stability case studies reported in the literature, there are very few authors who have calculated stability by both the TSA and ESA for the same embankment. This may be due to a bias toward one type of analysis by the authors and/or due to difficulties in using one type of analysis for a particular class of stability.

The following sections will compare the TSA and ESA of embankments on soft clay for three conditions: actual failures, variation in the factor of safety with embankment height and the long term stability.

5.1 CASES OF EMBANKMENT FAILURE

At failure conditions the value of the limiting equilibrium factor of safety, as defined in Chapter 2, is 1.0. By comparing the results of both types of analyses for failure conditions, the values of the factor of safety can be compared to the known value of 1.0 and not just to the other. For embankment conditions other than failure, the actual factor of safety is unknown and a comparison of the factor of safety from a TSA and an ESA would be only relative.

The 'end-of-construction' class of stability is the most critical class for embankments on soft clay and these problems have usually been treated using a TSA (Section 4.1). A review of embankment failures reported in the literature has produced eight cases which were analyzed by both types of analyses. These eight embankment failures were reported by a total of twelve researchers and are listed in Table 5-1. Two of the embankments, Portsmouth and I-95, were test sections intentionally constructed to failure. The Atchafalaya levee didn't fail by a rapid rotation or sliding motion but the undrained deformations were large and tension cracks appeared in the fill so that both researchers concluded that the embankment was at failure. The other five embankments experienced accidental failures during construction. Considering the large number of embankment

failures reported in the literature this small number of embankments analyzed with both types of analysis is very small and perhaps indicates how seldom the ESA is used for the "UU" class stability problem.

The TSA for these embankment failures are divided into six groups based upon the technique used to determine the undrained strength. These groups are: field vane, corrected field vane (Bjerrum, 1972), SHANSEP (Ladd and Foott, 1974), unconfined compression, unconsolidated-undrained triaxial compression tests and an average of the unconfined and "UU" triaxial compression tests. The ESA is divided into two groups based upon the method of determining the pore pressure, by measuring piezometer levels or predictions based on the loading conditions. All the results are given in Table 5-1 and shown graphically in Figure 5-1.

Both types of analyses show a wide fluctuation in the computed factors of safety and considering all data equally neither type seems to be very accurate. However some of the techniques used to measure the undrained strength are known to introduce a large source of error into the calculations. If these techniques and their resulting factors of safety are eliminated from the comparison the accuracy of the TSA is vastly improved and certainly superior to the ESA. The field vane strengths have been shown by Bjerrum (1972) to overestimate the shear strength of plastic soil

and the field vane strengths should not be used without applying Bjerrum's empirical correction factor. The corrected field vane values of the factor of safety are quite good except for the Portland Organic Clay. This clay had wood pieces and shells throughout the layer and the field vane results, even with the correction factor, would be expected to give unacceptably high results. The unconfined compression tests can also introduce large amounts of error due to sample disturbance and should not be considered accurate in any case. With all of these TSA test results eliminated from consideration and only the lower value of any range given by a author used in the comparison, the range of the TSA based on SHANSEP is 1.08 to 0.82 with an average of 0.98, the TSA based on corrected field vane tests is 1.30 to 0.99 with an average of 1.07 and the ESA ranges from 1.95 to 0.96 with an average of 1.27. These results from actual embankment failures plotted in Figure 5-2 support the use of TSA for the "UU" class of stability for embankments on soft clay.

5.2 VARIATION WITH EMBANKMENT HEIGHT

A comparison of the factor of safety calculated by both types of analyses at any condition other than failure is less meaningful because the actual factor of safety is unknown. Nevertheless the variation of the factor of safety with changes in the embankment height should show a trend

of decreasing factor of safety with increasing height. At the failure condition the factor of safety for both types of analyses should be 1.0.

To determine the variation of the factor of safety with embankment height of an embankment constructed to failure, the Portsmouth, New Hampshire Experimental Test Section (Ladd, 1972) was selected. The Portsmouth embankment was constructed on a soft sensitive clay and heavily instrumented so both a TSA and an ESA could be conducted. The soil properties, embankment geometry and stability analyses are given in Appendix A and will not be repeated here. The TSA was based on the average SHANSEP strength values computed for $\overline{CK_0U}$ tests for shear and oedometer tests for stress history. The ESA was based on measured pore pressures and the drained strength parameters, $\bar{\phi} = 21^\circ$ and $\bar{c} = 0$, which were the lower bound from $\overline{CIU}$ tests on the soil. The values of the factor of safety with changes in the embankment height are listed in Tables A-4 and A-5 of Appendix A and shown graphically in Figure 5-3. The factor of safety from an effective stress analysis is always higher than that from a total stress analysis but their values approach each other near the failure height.

In addition to the Portsmouth embankment, six other embankments are listed in Table 5-2 where the variation of both the TSA and ESA factor of safety with embankment height

were reported. The TSA for the four embankments reported by Tavenas et al. (1978) were computed using uncorrected field vane tests after each increment of loading for these stage constructed embankments. The TSA for the New Liskeard varved clay embankment were based on average unconfined and unconsolidated-undrained triaxial compression tests. The other embankments listed in Table 5-2 used the SHANSEP technique for calculating the TSA factor of safety. All of the embankments had piezometers to measure the pore pressure which were used to determine the ESA factor of safety. The results for these seven embankments are shown in Figure 5-4 which plots the TSA factor of safety versus the ESA factor of safety for different elevations of the embankments. In every case the ESA gives a higher value for the factor of safety than the TSA for the same height of embankment.

With increasing embankment height both factors of safety decrease with the ESA decreasing at a faster rate than for the TSA. Tavenas et al. (1978) predicted a rapid decrease in the effective stress analysis factor of safety after a portion of the clay foundation reached local yielding near a factor of safety of 1.3 (Figure 4-3). However, the embankments they analyzed were not loaded to failure and the behavior of the Portsmouth embankment (Figure 5-3) does not confirm this rapid decrease in the ESA factor of safety near 1.3. The safety factor from the ESA decreases at a

rate faster than the TSA but there does not appear to be any sudden change in the slope to indicate local yielding. D'Appoloia et al. (1972) showed that local yielding occurs at a factor of safety of four for normally consolidated clay and for heavily overconsolidated clay local yielding first occurs at a factor of safety of 1.8 or higher. The pore pressures measured a few hours after the failure of the Portsmouth embankment (Ladd, 1972) averaged twenty-five percent higher than the pore pressures measured a few hours before failure. These rapid pore pressure increases were due to the shear during failure and did not provide any indication of impending failure. The Interstate-95 embankment (Silva-Tulla et al. 1976) also showed increased pore pressure after failure but no warning of impending failure.

Of the seven embankments only the New Liskeard varved clay and the Portsmouth were built to failure. Their factors of safety were respectively: 1.00 and 0.90 for a TSA and 1.15 and 0.96 for the ESA.

5.3 LONG TERM STABILITY

The long term stability of an embankment is usually determined by an effective stress analysis using hydrostatic or steady state pore pressures. The long term stability determined by a total stress analysis is the same as a "CU" class problem with complete consolidation and no pore

pressure dissipation during shear. Although the long term class is never the critical condition for embankments on soft clays, a comparison of the factors of safety computed from the two types of analyses may lend insight into their use and applications.

The embankments considered are both hypothetical examples using the soil properties of two highly tested clays: Boston Blue Clay (BBC) and Connecticut Valley Varved Clay (CVVC). The BBC model embankment is described in Appendix B along with the techniques used to evaluate the factor of safety for TSA and ESA. The CVVC model embankment is described by Ladd and Foott (1977) and shown in Figure 5-5.

The end-of-construction stability was determined for a comparison with the long term stability. The CVVC model embankment had a "UU" factor of safety of 1.27 based on corrected field vane (Bjerrum, 1972) and Simplified Bishop method of slices. The BBC model had a "UU" factor of safety of 1.02 using SHANSEP strength with Simplified Bishop method and 0.94 using STAB3D analysis (Table 5-3). The BBC model embankment was constructed slowly to allow increased strength due to consolidation.

For the long term class the ESA uses the drained strength parameters for the soil, $\bar{c}$ and $\bar{\phi}$, and static pore pressures. One-dimensional consolidation was considered to obtain the settlement beneath the embankments but the crest

elevations remained constant. Both embankments were analyzed using the Simplified Bishop method of slices and have factors of safety of 2.40 and 2.80 for the BBC and CVVC respectively. The ESA procedure is straight forward and represents common practice for determining the long term stability.

To compute the long term stability using a TSA the embankments were divided into zones and the average overburden of each zone was used to compute the average effective consolidation stress. For example, the BBC model was divided into three zones: centerline to crest, crest to toe and beyond the toe. These zones also represent the active, direct and passive regions for shear strength. Using the average effective stresses computed in this was the SHANSEP technique was applied by computing the strength for Plane Strain Compression (PSC), Direct-Simple Shear (DSS) and Plane Strain Extension (PSE) test results for the appropriate zone. The strengths used for both embankments were the strengths on the failure surface at failure (τ_{ff}), not the maximum shear q_f, and both were corrected for strain compatibility.

The CVVC embankment was analyzed by the Morgenstern-Price (1965) method of slices and the resulting minimum factor of safety was 1.50. The BBC embankment was analyzed by the Simplified Bishop method of slices and the resulting factor of safety was 1.40. The STAB3D method of slices

uses the anisotropic strength properties and inclination of the failure surface at each slice to determine the shear strength. The long term factor of safety for the BBC model using the STAB3D method was 1.14. The difference between the Active, Direct and Passive (ADP) method and the STAB3D method is in the strength beneath the slope where the Direct Simple Shear strength is higher than the STAB3D strength (Figure B-7). The factors of safety are summarized in Table 5-3.

The estimation of the SHANSEP undrained strengths may be conservative because they are based on the K_o stress conditions during consolidation and not the in situ stress. Ladd and Edgers (1972) showed the effects of rotating the principle stress during consolidation on $\overline{\text{CAU}}$ DSS tests. The differences in the two 'types' of analyses for the long term class of stability problem are still dramatic and represent a significant difference between the two approaches. Since the long term class is not the critical class for stability the difference is not of practical importance unless a rapid undrained loading condition can be expected.

TSA vs ESA FACTOR OF SAFETY, ACTUAL EMBANKMENT FAILURES

EMBANKMENT & REFERENCE	PI	TSA FV ○	TSA μFV ●	TSA SHANSEP ■	TSA UC ▽	TSA UU △	TSA AVE. ▲	ESA PIEZ	ESA EST.
1 New Liskeard Varved Clay	25								
a Lacasse & Ladd, 1973		1.12	0.99	1.05			1.00	1.05	
b Lo & Stermac, 1965							0.85*	1.20	
2 Portsmouth, N.H.	15								
a Ladd, 1972		0.88		1.01	0.70			1.08-1.14	
b Boyce, 1978				0.90				0.96	
3 Atchafalaya Levee	80								
a Foott & Ladd, 1977		1.92	1.30	0.97				1.56	
b Kaufman & Weaver, 1966							1.11	1.00	
							1.07	1.05	
4 Scottsdale	108								
Parry & MacLeod, 1967		1.65	1.02						1.95
5 Portland Maine Organic	32								
Ladd et al. 1969 OR		2.06	1.85	0.82	0.71	0.94			1.25
Gordon, 1973									
6 Scrapsgate	82								
a Golder & Palmer, 1955		1.3-				1.00			1.3-
b Bjerrum, 1972		1.52	1.03						
7 I-95, Revere, Mass.	20								
Silva-Tulla et al. 1976		1.06	1.03	1.03				1.16	
8 I-77 & I-80, Ohio	20								
a Wu, Thayer & Lin, 1975				1.08-1.28				1.13-1.46	
b " " " "					1.25-1.44			1.55-1.95	

* Average of UC, UU and FV.

TABLE 5-1

TSA vs ESA FACTOR OF SAFETY
VARIATION WITH EMBANKMENT HEIGHT

EMBANKMENT & REFERENCE	HEIGHT FILL	TSA	ESA
1 Rang St-George Tavenas et al. 1978	7.4m 8.3m	1.32 1.25	2.08 1.86
2 Rang du Fleuve Tavenas et al. 1978	5.7m 6.7m 7.8m	1.73 1.45 1.24	2.03 1.60 1.35
3 Rang du la Concession Tavenas et al. 1978	4.8m 6.6m 8.1m	1.85 1.66 1.40	2.45 2.08 1.75
4 Rang du Brulé Tavenas et al. 1978	6.3m 8.8m	2.05 1.55	2.3 1.7
5 Connecticut Valley Connell 1972	20.0ft 36.7ft	2.34 1.46	2.79 1.76
6 New Liskeard Varved Clay Lacasse & Ladd 1973	18.0ft 20.0ft	1.20 1.00*	1.28 1.05*
7 Portsmouth N.H. Boyce 1978	10.0ft 13.0ft 15.5ft 18.0ft 19.5ft 20.5ft 21.5ft	1.88 1.42 1.19 1.05 0.98 0.94 0.90*	2.50 1.88 1.53 1.29 1.14 1.03 0.96*

* Embankment failure.

TABLE 5-2

TSA vs ESA FACTOR OF SAFETY

MODEL EMBANKMENTS

END-OF-CONSTRUCTION*

METHOD		BBC	CVVC
TSA	DSS Simplified Bishop ADP Simplified Bishop STAB3D FV Simplified Bishop	1.02 1.02 0.94	 1.27

LONG TERM

METHOD		BBC	CVVC
ESA	Simplified Bishop	2.40	2.80**
TSA	ADP Simplified Bishop Morgenstern-Price STAB3D	1.40 1.14	 1.50

BBC = Boston Blue Clay

CVVC = Connecticut Valley Varved Clay

DSS = Direct-Simple Shear

ADP = Active, Direct and Passive zones.

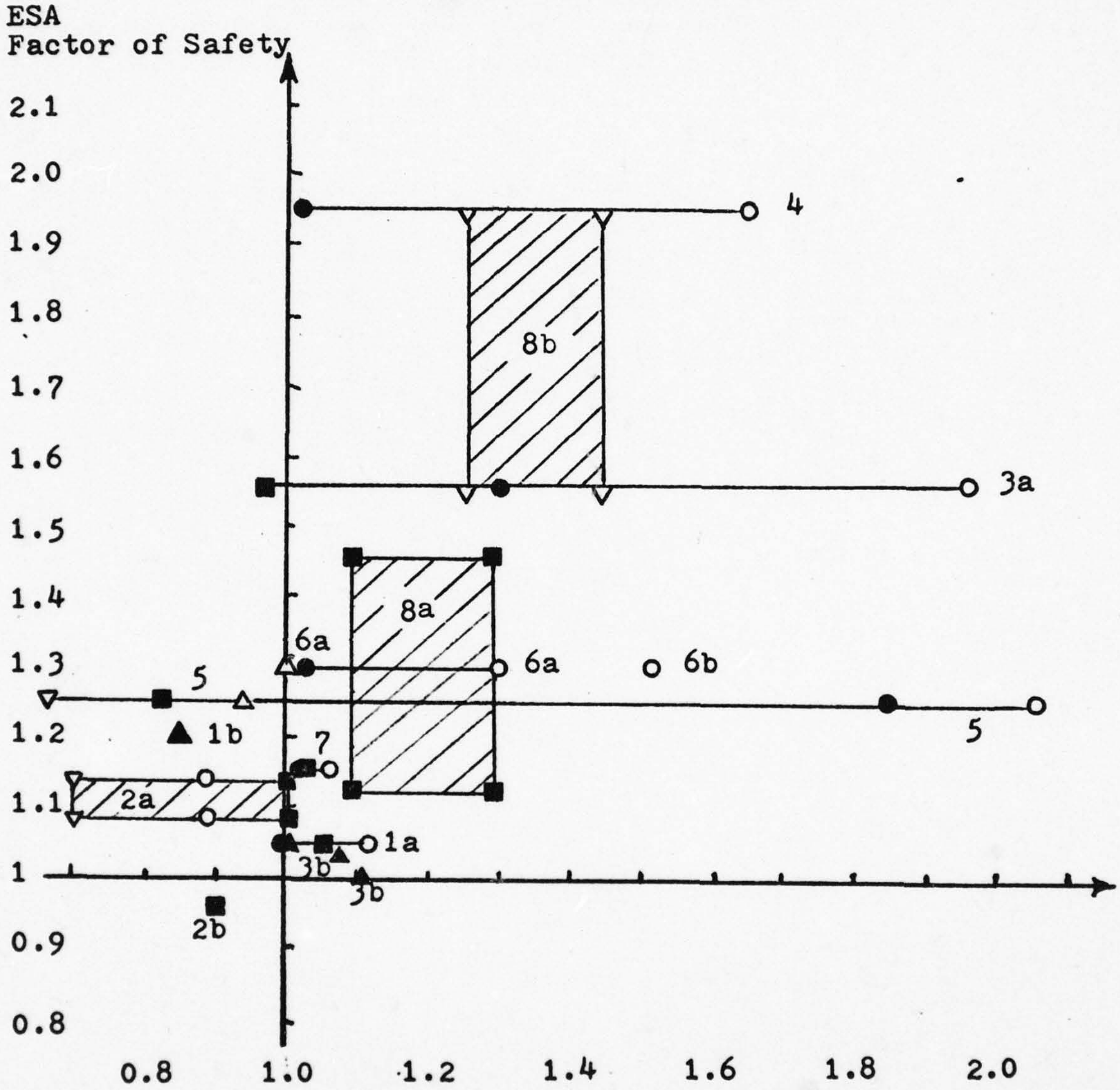
* Does not include height of fill required to compensate for consolidation settlements.

** Reasonable failure surface. Minimum occurred in berm without passing thru clay.

TABLE 5-3

ACTUAL EMBANKMENT FAILURES

TSA vs ESA FACTOR OF SAFETY*



○ Field Vane
● Corrected Field Vane
■ SHANSEP
▽ Unconfined Compression
△ Triaxial "UU"
▲ Average of ▽ and △

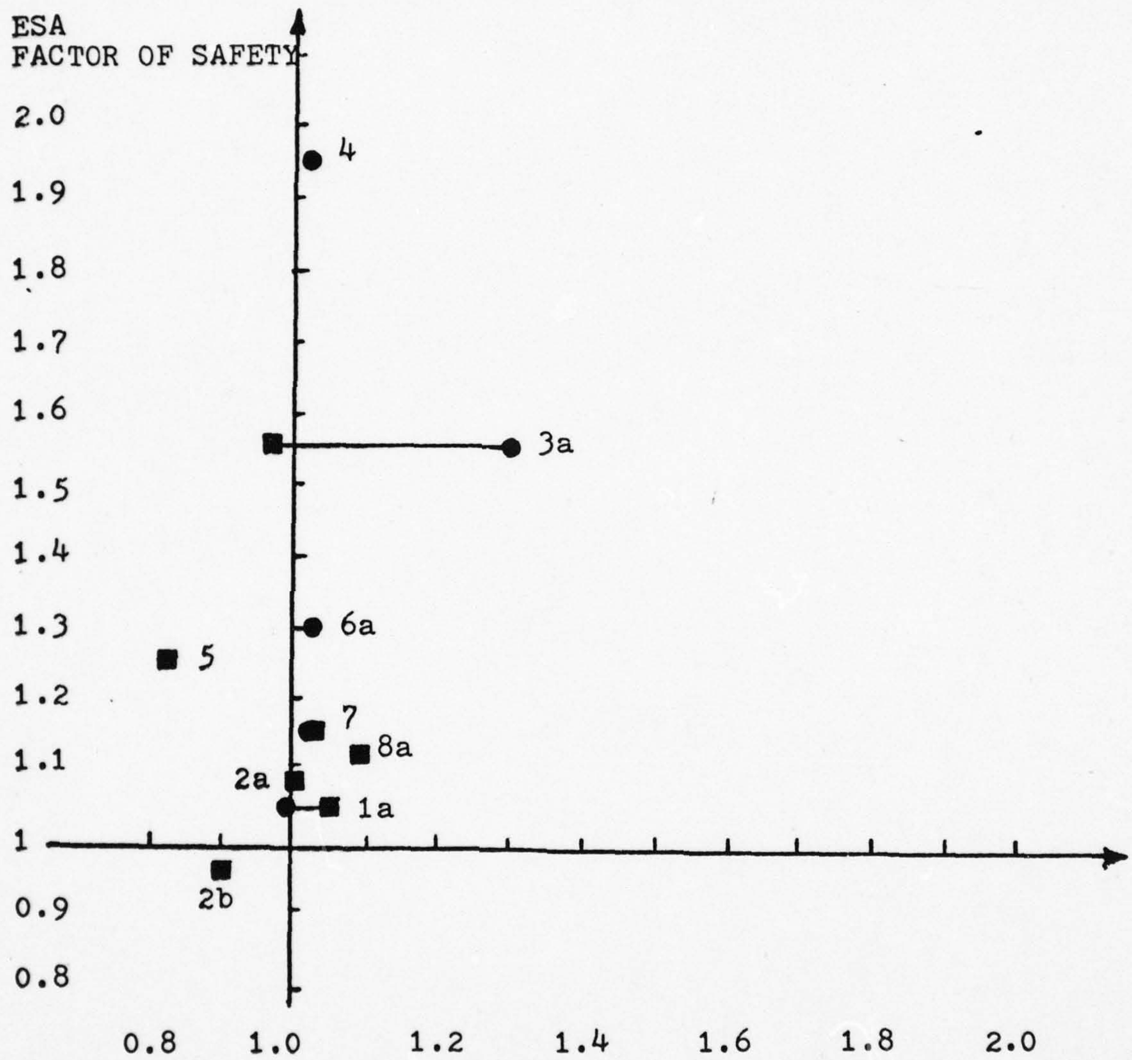
* Values listed in Table 5-1

FIGURE 5-1

ACTUAL EMBANKMENT FAILURES

TSA vs ESA FACTOR OF SAFETY*

CORRECTED FIELD VANE & SHANSEP

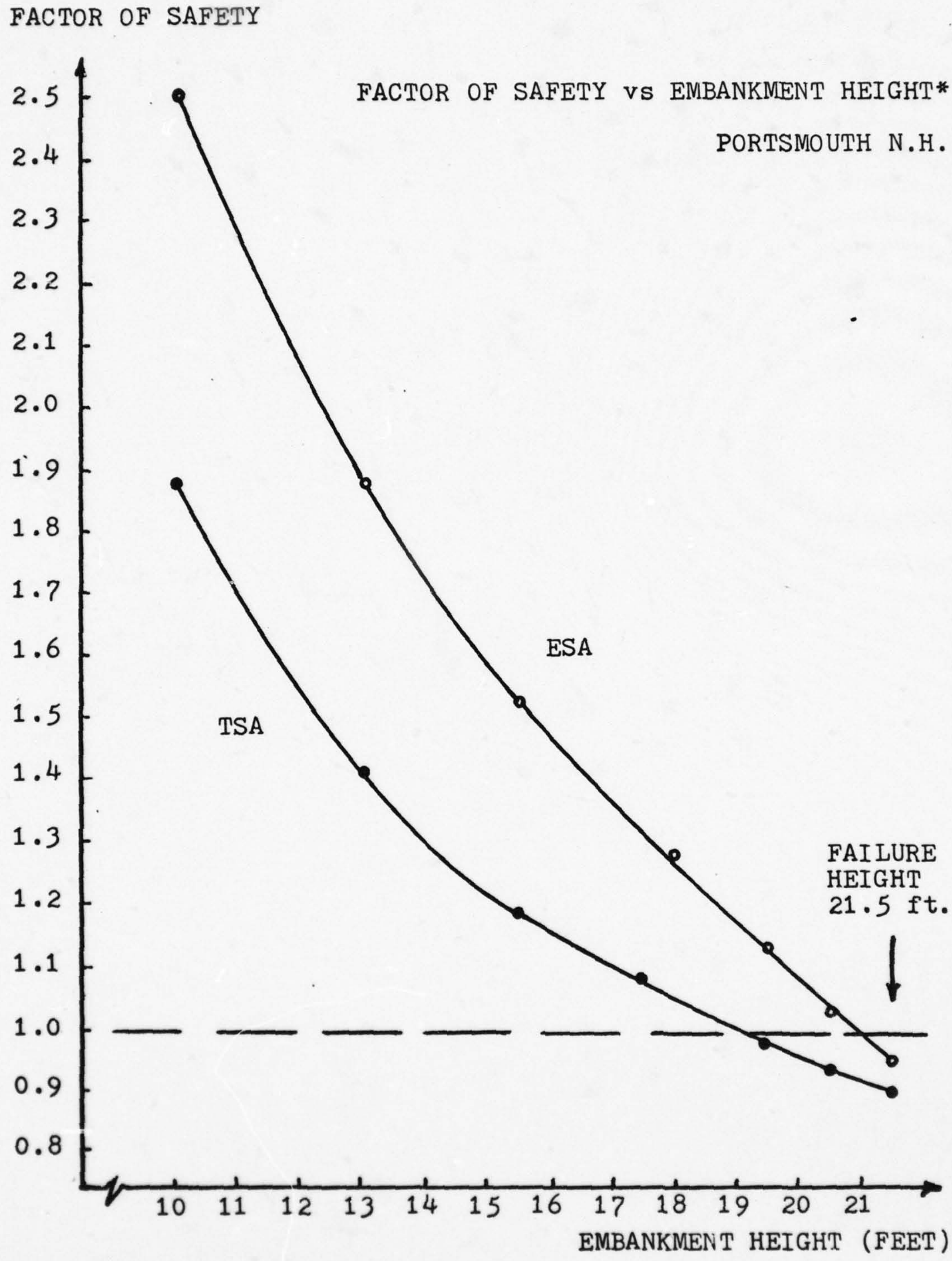


● Corrected Field Vane (Bjerrum, 1972)

■ SHANSEP (Ladd and Foott, 1974)

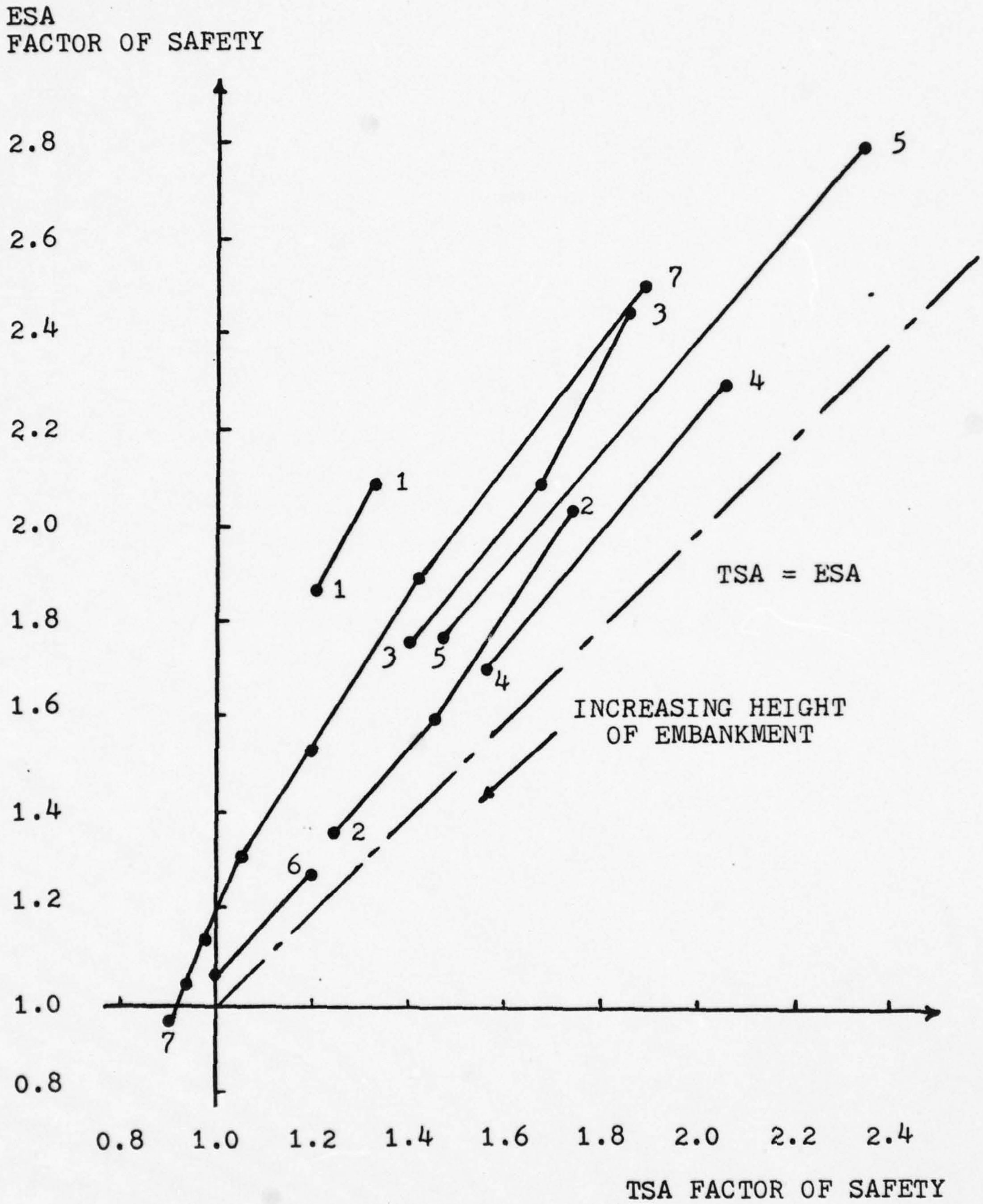
* Values listed in Table 5-1

FIGURE 5-2



* Values listed in Tables A-4 and A-5 of Appendix A.

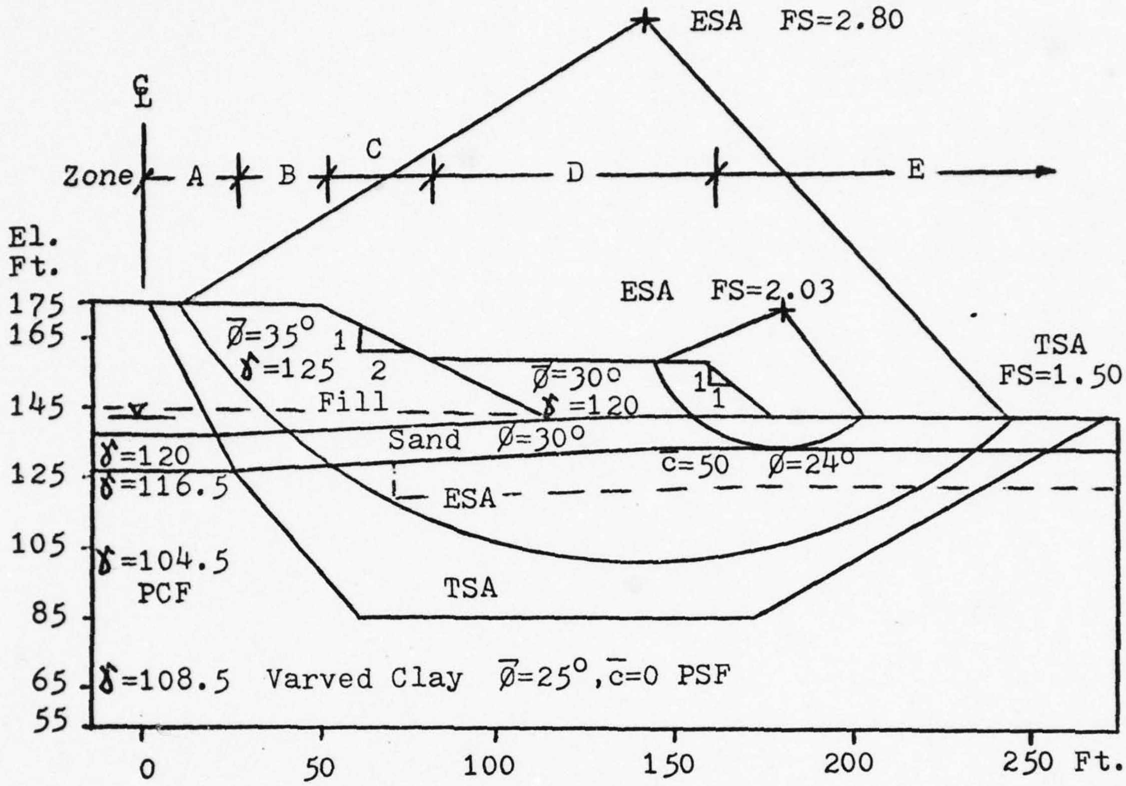
FIGURE 5-3



TSA vs ESA FACTOR OF SAFETY
WITH VARIATION IN EMBANKMENT HEIGHT*

* Values listed in Table 5-2.

FIGURE 5-4



BEFORE CONSOLIDATION

METHOD	F.S.
TSA Simplified Bishop μFV	1.27

AFTER CONSOLIDATION

METHOD	F.S.
ESA Simplified Bishop	2.80
TSA Morgenstern-Price SHANSEP	1.50

CONNECTICUT VALLEY VARVED CLAY

MODEL EMBANKMENT

FIGURE 5-5

CHAPTER 6

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 SUMMARY AND CONCLUSIONS

This thesis considers the stability analysis of embankments constructed on soft clays. The term 'soft' clay implies that the embankment loading exceeds the preconsolidation stress in the soil and the clay becomes normally consolidated. 'Methods' of stability analysis, 'types' of stability analysis and 'classes' of stability problems are defined and discussed.

The 'methods' of analysis are the techniques used to determine the limiting equilibrium factor of safety (Equation 2-1) and involve the selection of the critical failure surface and the assumptions necessary to make statically determinate the equations of equilibrium (Table 2-1). Most methods of analysis are reasonably accurate, less than ten percent difference between them, and the choice of methods is up to the engineer based on one's own experience, shape of the assumed failure surface and technique (arthimetic, graphical or computer) available for solution.

The 'type' of analysis distinguishes between the choice of expressions for calculating the shear strength of the soil. The total stress analysis (TSA) considers the shear strength

on the failure surface at failure to be independent of the changes in total normal stress. The strength may vary with stress history, inclination of the failure surface and level of strain. The effective stress analysis (ESA) uses the Mohr Coulomb failure criteria (Equation 3-1) to express the strength in terms of the effective strength parameters ($\bar{c}$ and $\bar{\phi}$), the total normal stress on the failure surface (σ) and the pore pressure (u).

The accuracy of either 'type' of analysis is directly related to the ability to measure the strength parameters. The TSA requires the single parameter of undrained strength. In the past the $\phi=0$ analysis was used to obtain a single value (q_f) for the undrained strength for a given effective consolidation stress or natural water content. This technique is not considered accurate and in this thesis the value of the shear stress on the failure surface at failure (τ_{ff}) with consideration of the anisotropic properties of clays and the effects of strain levels at failure, i.e. strain compatibility, are used. Common strength tests such as the field vane, unconfined compression and "UU" triaxial compression tests frequently do not give accurate results and thus an empirical correction factor for the field vane (Bjerrum, 1972) and/or more rational lab testing techniques, i.e. SHANSEP, which rely on normalized soil behavior, are considered here.

The ESA strength parameters, $\bar{c}$ and $\bar{\phi}$, are measured in laboratory tests and are subject to numerous testing errors e.g. sample disturbance and anisotropy, as well as test interpretation difficulties e.g. use of maximum shear stress, maximum obliquity or line of tangency to determine the friction angle. The total normal stress is usually determined by the 'method' of analysis through some assumptions to make the equations of equilibrium statically determinate. The most difficult parameter to predict for the ESA during the design is the pore pressure which may only be crudely approximated. During construction the pore pressures can be measured with piezometers but even then the ESA does not include the increased pore pressures generated during shear.

Three 'classes' of stability problems are defined, based upon the drainage conditions during loading and shear to failure. These classes are labeled after similar lab tests: "UU" = unconsolidated-undrained or end-of-construction, "CD" = consolidated-drained or long term and "CU" = consolidated-undrained or intermediate stability problems. In theory, each class of stability could be solved with either type of analysis i.e. TSA or ESA. However, in practice on type of analysis is usually better suited for a particular class of problem.

The TSA and ESA factors of safety are compared for three cases: actual embankment failures, with variation in

embankment height and long term stability. The end-of-construction class is the most critical for embankments on soft clays and are usually treated by a TSA. Only a limited number of cases could be found for actual embankment failures in which both types of analyses were used. These results are listed in Table 5-1 and plotted in Figure 5-1. Although neither the TSA or the ESA appears very accurate in predicting failure conditions considering all data equally, but when only the more accurate techniques for determining the undrained strength, i.e. corrected field vane and SHANSEP, are used for the comparison (Figure 5-2), the TSA is far superior to the ESA for the "UU" class.

The factor of safety decreases with increasing embankment height for both types of analyses with both reaching a factor of safety near 1.0 at failure. For the cases reported in the literature (Table 5-2) and the Portsmouth, N.H. embankment analyzed in Appendix A, the ESA factor of safety was higher than the TSA at all embankment heights (Figure 5-4). The ESA factor of safety decreases more rapidly near failure than the TSA but there was no sudden change as predicted by Tavenas et al. (1978) (Figure 4-3) nor was there any rapid increase in piezometer levels to indicate impending failure. Several embankments, Portsmouth (Ladd, 1972) and I-95 (Silva Tulla et al. 1976), have experienced rapid increases in pore pressures after failure due to gross shear movements but in

neither case were there any indications of imminent stability failure. The differences in the factors of safety are due to the increased pore pressures during shear considered in the TSA but neglected in the ESA. The difference does not imply error in either type of analysis nor does it imply any difference in the stability of the embankment, but only the difference in strength expressions used in the analyses.

The long term stability for both types of analyses was considered for two hypothetical embankments constructed on highly tested soils: Connecticut Valley Varved Clay (Ladd and Foott, 1977) and Boston Blue Clay (Appendix B). The ESA for the long term class uses the drained strength parameters, $\bar{c}$ and $\bar{\phi}$, and static or steady state pore pressures and is the approach most used in practice. The TSA for the long term class is the same as a "CU" class problem with complete consolidation. In both cases the ESA factor of safety was significantly higher than the TSA (Table 5-3) and again demonstrates the effects of neglecting the pore pressures generated during shear to failure.

6.2 RECOMMENDATIONS

The following recommendations are made for the stability analysis of embankments constructed on soft clay.

The end-of-construction class of stability is the most critical for embankments on soft clay. Design for this class

of stability is best conducted using a total stress analysis with undrained strength determined by corrected (Bjerrum,1972) field vane or normalized soil properties such as SHANSEP (Ladd and Foott, 1974). Strength based on uncorrected field vane test, unconfined compression or "UU" triaxial tests are subject to large errors and may give incorrect results.

For field monitoring of the construction, particularly for stage constructed embankments, either type of analysis may be used. The TSA must be based on normalized soil properties or actual measurements by in situ testing or sampling for each stage of construction. The ESA may be based on measured piezometer levels and the factor of safety at any instant of time may be determined. The ESA is easier to apply in this case but does not include the effects of pore pressures generated during shear and will always give a higher value for the factor of safety than the TSA.

The long term stability is never the critical class for embankments on soft clay and therefore stability is not a problem. The easiest type of analysis to apply is the ESA based on static or steady state pore pressures, but the results form the ESA will be significantly higher than the factor of safety from a TSA which includes the pore pressures generated during shear.

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APPENDIX A

PORTSMOUTH, NEW HAMPSHIRE TEST EMBANKMENT

This section analyzes the construction to failure of an embankment located on a soft sensitive clay. The embankment was the Experimental Test Section of Interstate 95 in Portsmouth, N.H., which was an extensively instrumented embankment loaded to failure by rapid construction. The field data are used for two comparisons: the factor of safety by total stress and effective stress analyses at failure and the variation in the factor of safety with embankment height.

The Portsmouth Experimental Test Section was selected because of the extensive instrumentation which allowed both a total stress and effective stress analysis to be conducted. This embankment failure was reported by Ladd (1972) and additional unpublished data were available at M.I.T. The description of soil conditions, testing procedures and field data are presented in Ladd (1972) and will not be repeated here.

The soil properties, embankment geometry and location of the failure surface are shown in Figures A-1 and A-2. Because the observed failure surface occurred along a circular arc, the Simplified Bishop method of slices (Bishop, 1955) was determined to be a reasonable method for stability

analysis (Wright, 1969)(Wright et al. 1969). The stability analysis was performed on LEASE II (Dawson, 1972), a subsystem of ICES, Integrated Civil Engineering System, available at M.I.T.

The total stress analysis was performed on this 'end-of-construction' class problem using the undrained strength determined by the SHANSEP (Stress History And Normalized Soil Engineering Properties) method with $\overline{CK_0U}$ Direct-Simple Shear test data and the average preconsolidation stress determined from oedometer tests. The strength values are shown in Figure A-1 and listed in Table A-2 under soil data. The values of the TSA factor of safety with variation in the embankment height and at failure are shown in Figure 5-3 and listed in Table A-4. Run number 12 was considered to be the most accurate and was one percent lower than run 11. Values from run 11 were reduced one percent to reflect this decrease when plotted in Figure 5-3. The factor of safety for the observed failure surface is the same as reported by Ladd (1972) but the critical Bishop factor of safety is lower, 0.90 compared to 1.01. This lower factor of safety is due to the location of a different relative minimum and not due to any changes in geometry or strength. This difference points out the difficulty of evaluating stability when several relative minimums may be calculated.

The effective stress anaylsis was performed using

measured piezometer readings throughout the foundation clay (Figure A-2). The soil profile was divided into layers and a piezometric line was constructed for each layer based on the field readings. The layers included: a top layer of sand with static water table, a clay crust assigned the same values of undrained strength as the total stress analysis due to the difficulties in measuring pore pressures in this overconsolidated crust, a sand silt layer with measured pore pressures, a clay layer with pore pressures averaged between the soft clay and the sand silt layers (Figure A-1). The intermediate clay layer was selected because of the lateral spreading of the pore pressures in the confined sand silt layer. The piezometric line data for the layers described above are given in Table A-3 with variation in embankment height. The strength parameters for the effective stress analyses are shown in Figure A-1 and listed in Table A-2. The friction angle for the soft clay of 21° corresponds to the value at maximum shear for $\overline{\text{CUC}}$ tests and gives a factor of safety of 0.96 for the critical Bishop surface. A friction angle of 25° would be a more reasonable average for this soft clay but results in a factor of safety of 1.11 for the minimum value, an increase of fifteen percent. The factor of safety for variation with embankment height and at failure are given in Table A-5.

To compare the strength along the observed failure

surface using the two types of analyses, i.e. TSA and ESA, the embankment was divided into slices as shown in Figure A-3. These slices were drawn to intercept a single layer of soil as used in the total stress analysis. The stability of this observed failure surface was computed manually and the average normal stress was used to compute the average shear strength along the arc of each slice. The value of the factor of safety for the ESA by both LEASE II and manually was 1.16 and for the TSA was 1.01. The average shear strengths along the failure surface are computed in Tables A-6 and A-7 and shown graphically in Figure A-3. The ESA gave higher values of strength to the soil beneath the crest and slope and lower values of strength for the soil at the toe than did the TSA.

INPUT GEOMETRY

PORTSMOUTH N.H. EMBANKMENT

POINT DATA (FEET)		
No.	X	Y
1	0	-14
2	200	-14
3	0	-10
4	200	-10
5	0	-5
6	200	-5
7	0	-2.5
8	200	-2.5
9	0	0
10	200	0
11	0	5
12	200	5
13	0	10
14	200	10
15	0	15
16	200	15
17	0	20
18	50	20
19	200	20
20	0	22
21	58	22
22	200	22
23	102	33
24	107.5	35.5
25	114	39.5
26	115	41.5
27	145	41.5
28	146	39.5
29	150	35.5
30	158	33
31	176	33
32	200	30

LINE DATA	Point		
No.	Left	Right	Soil#
1	1	2	1
2	3	4	2
3	5	6	3
4	7	8	4
5	9	10	5
6	11	12	6
7	13	14	7
8	15	16	8
9	17	18	9
10	18	19	9
11	20	21	10
12	21	22	11
13	18	21	11
14	21	23	12
15	23	24	12
16	24	25	13
17	25	26	13
18	26	27	13
19	27	28	13
20	28	29	13
21	29	30	12
22	30	31	12
23	31	32	12
24	24	29	12

TABLE A-1

SOIL PARAMETERS

PORTSMOUTH N.H. EMBANKMENT

SOIL DATA					
		TOTAL STRESS		EFFECTIVE STRESS	
No.	Unit Weight (PCF)	C (PSF)	Ø (°)	C (PSF)	$\bar{Ø}$ (°)
1	130	0	30	0	30
2	120	400	0	0	21
3	120	355	0	0	21
4	109	320	0	0	21
5	109	300	0	0	21
6	109	260	0	0	21
7	109	230	0	0	21
8	118	335	0	335	0
9	118	1000	0	1000	0
10	62.4	0	0	0	0
11	130	0	40	0	40
12	115	0	40	0	40
13	110	0	30	0	30

TABLE A-2

PIEZOMETER LINE DATA
PORTSMOUTH N.H.

PIEZOMETER LINE No. SOIL LAYER	X (FEET)	EMBANKMENT CREST ELEVATION (FEET) 41.5	40.5	39.5	38	35.5	33	30
		Y (FEET)						
1 Water Table	0	22	22	22	22	22	22	22
	200	22	22	22	22	22	22	22
2 Sand Silt El -14 -20	0	22	22	22	22	22	22	22
	60	28	27.5	27	26	25	24	23
	110	45.5	44	43	41	38	34.5	27.5
	150	45.5	44	43	41	38	34.5	27.5
	200	30	29.5	29	28.5	28	27.5	27
3 Clay El -10 -14	0	23.5	23	23	23	22.5	22	22
	60	31	30	29.5	28	27	25	24
	110	46	45	44	42	39	35	29
	130	48	47	46	44	41	37	31
	150	46	45	44	42	39	35	29
	200	37	36	35	34	32	31	27
4 Clay El +10 -10	0	25	24.5	24	23.5	23	22.5	22
	60	34	33	32	30	28	26	24.5
	110	48	47	45.5	43.5	40.5	35	30.5
	130	50	49	47.5	45.5	42.5	37	32.5
	150	48	47	45.5	43.5	40.5	35	30.5
	200	39	38	37	36	34	31	28

TABLE A-3

TOTAL STRESS ANALYSIS vs EMBANKMENT HEIGHT

PORTSMOUTH N.H.

Crest El. (Feet)	Factor of Safety		Failure Surface			RUN#
	Actual Surface	Critical Bishop	X	Y (Feet)	R	
41.5	1.062	.908	88.75	47.50	58.18	9
		.903	90.00	45.00	55.67	10
	1.022	.909	88.75	45.00	56.73	11
	1.012		85.00	51.00	61.00	12
40.5	1.101	.952	85.00	42.50	51.32	9
	1.059	.949	88.75	45.00	56.35	10
	1.059	.949	85.00	47.50	60.84	11
	1.049		85.00	51.00	61.00	12
39.5	1.152	.997	85.00	47.50	55.00	9
	1.109	.985	90.00	45.00	55.67	10
	1.109	.990	88.75	45.00	56.73	11
37.5	1.266	1.084	85.00	47.50	55.00	9
	1.219	1.090	90.00	45.00	55.67	10
	1.219	1.091	85.00	55.00	60.00	11
35.5	1.509	1.196	85.00	40.00	50.77	10
	1.499	1.196	85.00	46.25	54.42	11
33.0	1.794	1.427	85.00	46.25	54.41	10
	1.778	1.428	85.00	46.25	54.42	11
30.0	2.408	1.911	80.00	45.00	50,00	10
	2.385	1.882	81.25	45.00	50.50	11

RUN#	Soil #11 Unit Wt. (PCF)	Toe Water Treatment C=Ø=0	Y Min (Ft.)
9	115	Yes	-14
10	115	No	-14
11	115	No	-20
12	130	No	-14

TABLE A-4

EFFECTIVE STRESS ANALYSIS vs EMBANKMENT HEIGHT

PORTSMOUTH N.H.

CREST El. (FEET)	Factor of Safety		Failure Surface			RUN#
	Actual Surface	Critical Bishop	X	Y	R	
41.5	1.156	.985	81.25	45.00	50.00	14
		.995	85.00	45.00	46.68	17
		.963	80.00	50.00	52.56	18
40.5	1.230	1.076	80.00	45.00	50.00	14
		1.032	80.00	51.25	53.85	17
		1.088	85.00	40.00	43.55	18
39.5	1.341	1.141	81.25	45.00	50.00	15
		1.173	85.00	41.25	44.79	18
38.0	1.519	1.291	81.25	45.00	50.00	15
		1.326	85.00	41.25	44.79	18
35.5	2.013	1.575	80.00	45.00	50.00	15
		1.531	80.00	51.25	53.80	18
33.0	2.563	2.017	80.00	45.00	50.00	15
		1.882	77.50	43.75	46.46	18
30.0	3.504	2.505	65.00	41.25	48.36	15
41.5	1.316	1.112	85.00	45.00	46.68	17*

RUN #	Surface Depth Min (ft)	Max	Ø Soft Clay
14	-5	-14	21°
15	-5	-14	21°
17	+10	-14	21°
17*	+10	-14	25°
18	0	-10	21°

TABLE A-5

MANUAL EFFECTIVE STRESS ANALYSIS, PORTSMOUTH N.H. (I)

SLICE	1 θ	2 ΔX	3 W	4 U	5 Wsinθ	6 CΔX
1	-56.6	3.3	1.49	0	-1.24	3.30
2	-51.3	4.0	4.04	0	-3.15	1.34
3	-40.5	11.7	21.64	26.85	-14.05	0
4	-28.3	9.3	25.83	35.95	-12.25	0
5	-18.6	9.2	31.96	42.05	-10.19	0
6	-9.6	10.0	40.38	47.15	-6.73	0
7	0	10.0	44.28	50.80	0	0
8	+9.6	10.0	46.13	52.75	+7.69	0
9	18.6	9.2	43.19	53.04	13.78	0
10	28.3	9.3	44.79	50.82	21.23	0
11	40.5	11.7	49.11	44.42	31.89	0
12	51.3	4.0	13.43	0	10.48	1.34
13	56.6	3.3	9.13	0	7.62	3.30
14	70.3	7.7	10.82	0	10.19	0

Σ 55.27

SLICE	7 UΔX	8 W-UΔX	9 8tan$\bar{\phi}$ *	10 6+9	11 M(θ) F=1.16	12 10÷11
1	0	0	0	3.30	.550	6.00
2	0	0	0	1.34	.625	2.14
3	19.60	2.04	.78	.78	.544	1.43
4	20.86	4.97	1.91	1.91	.723	2.64
5	24.14	7.82	3.00	3.00	.842	3.56
6	29.42	10.96	4.21	4.21	.931	4.52
7	31.70	12.58	4.83	4.83	1.000	4.83
8	32.92	13.21	5.07	5.07	1.041	4.87
9	30.45	12.74	4.89	4.89	1.054	4.64
10	29.49	15.30	5.87	5.87	1.038	5.66
11	32.43	16.60	6.40	6.40	.976	6.56
12	0	0	0	1.34	.625	2.14
13	0	0	0	3.30	.550	5.99
14	0	10.82	9.08	9.08	1.020	8,89

Σ 64.24

$$\text{Factor of Safety} = \frac{64.24}{55.27} = 1.16$$

* $\bar{\phi} = 21^{\circ}$

TABLE A-6

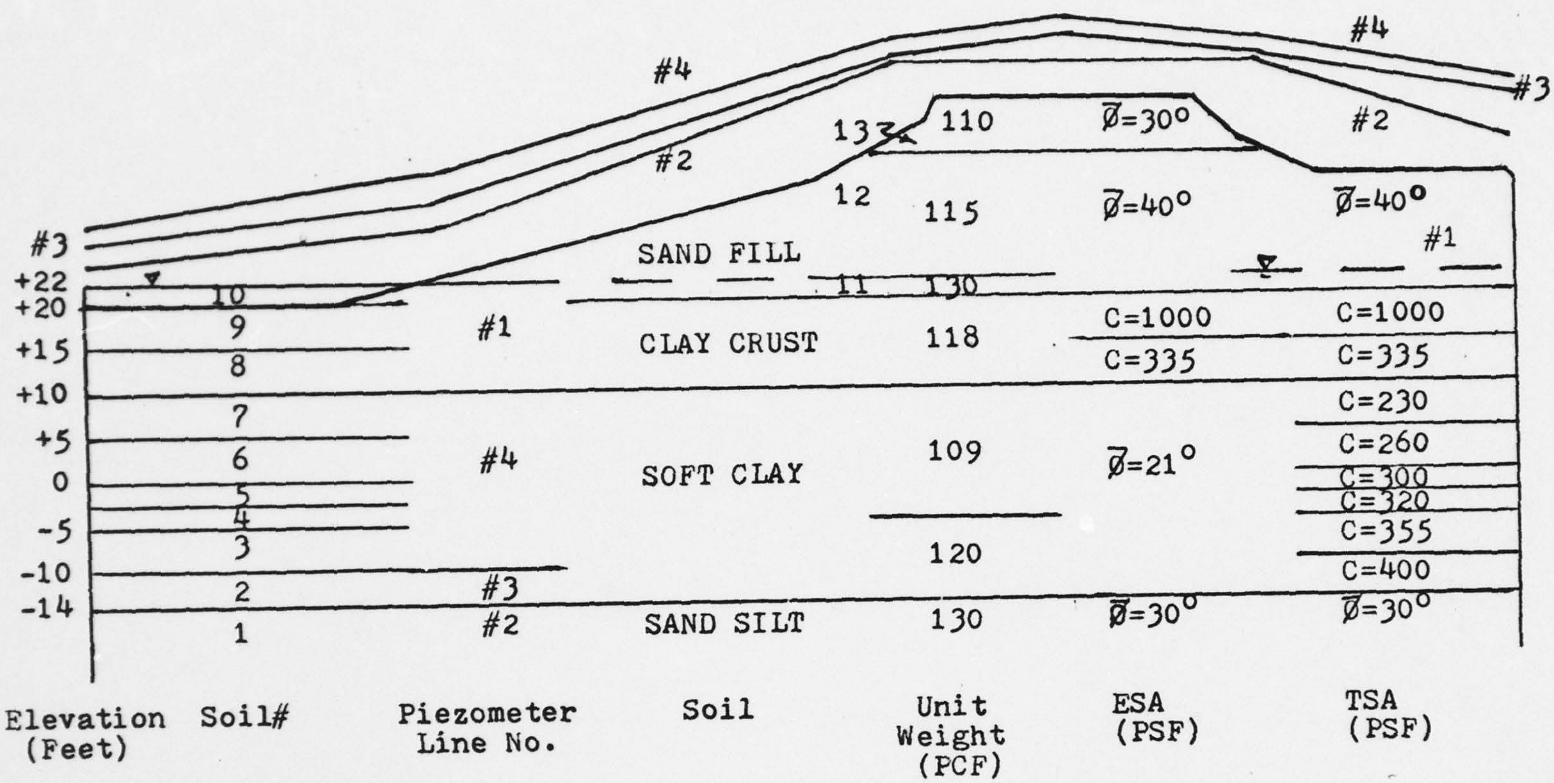
MANUAL EFFECTIVE STRESS ANALYSIS, PORTSMOUTH N.H. (II)

$$\bar{N} = \frac{W - U\Delta X - 1/F(\cancelto{0}{\bar{c}}\Delta X\tan\theta)}{\cos\theta(1 + 1/F(\tan\theta\tan\phi))} = \frac{W - U\Delta X}{M(\theta)}$$

$$S_d = \cancelto{0}{\bar{c}} + \frac{N\tan\phi}{L}$$

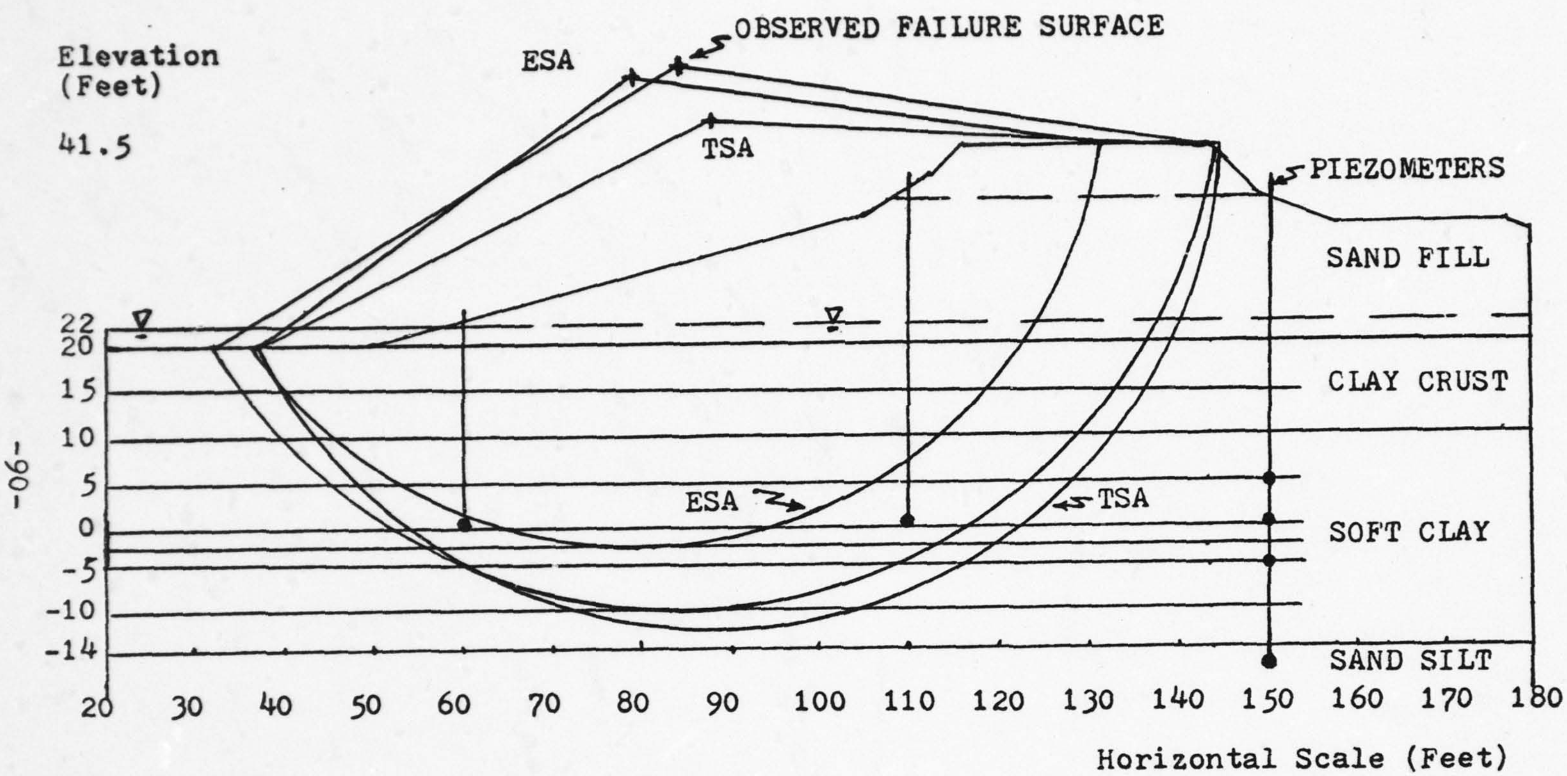
SLICE	W-UΔX	M(θ)	N	L	S_d	S_u
1					1000	1000
2					335	335
3	2.04	.544	3.75	15.39	94	245
4	4.97	.723	6.87	10.56	250	310
5	7.82	.842	9.29	9.71	367	355
6	10.96	.931	11.77	10.14	446	355
7	12.58	1.000	12.58	10.00	483	355
8	13.21	1.041	12.69	10.14	480	355
9	12.74	1.054	12.09	9.71	478	355
10	15.30	1.038	14.74	10.56	536	310
11	16.68	.976	17.09	15.39	426	245
12					335	335
13					1000	1000
14	10.82	1.020	10.61	17.31	514	514
				Factor of Safety	1.16	1.01

TABLE A-7



SOIL PROPERTIES
PORTSMOUTH N.H.

FIGURE A-1



CROSS SECTION OF EMBANKMENT FAILURE

PORTSMOUTH N.H.

FIGURE A-2

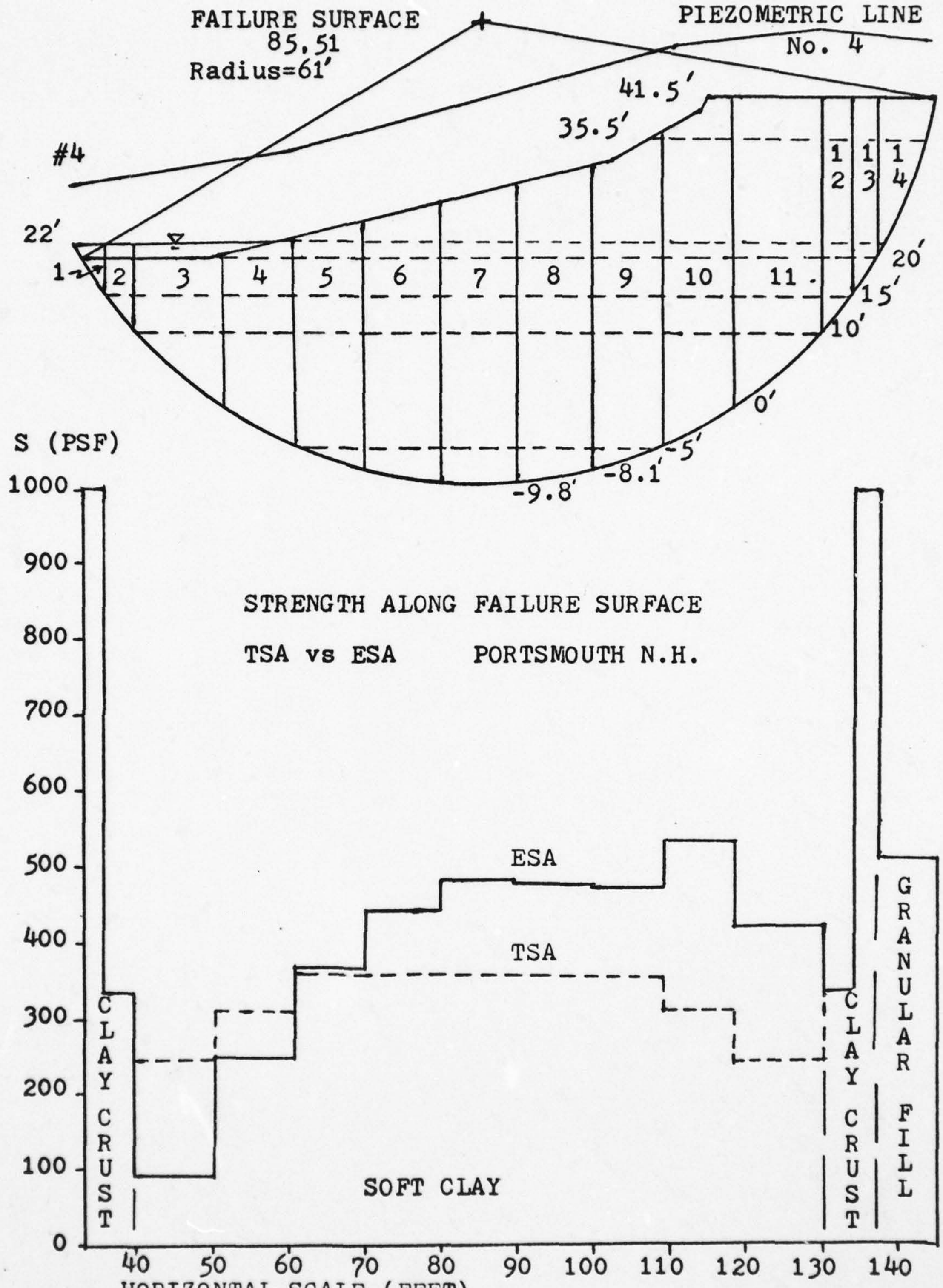

FAILURE SURFACE
85,51
Radius=61'
PIEZOMETRIC LINE
No. 4
#4
41.5'
35.5'
22'
1
2
3
4
5
6
7
8
9
10
11
12
13
14
20'
15'
10'
0'
-5'
-8.1'
-9.8'
S (PSF)
1000
900
800
700
600
500
400
300
200
100
0
STRENGTH ALONG FAILURE SURFACE
TSA vs ESA PORTSMOUTH N.H.
ESA
TSA
CLAY CRUST
SOFT CLAY
CLAY CRUST
GRANULAR FILL
40
50
60
70
80
90
100
110
120
130
140
HORIZONTAL SCALE (FEET)

FIGURE A-3

APPENDIX B

MODEL EMBANKMENT ON BOSTON BLUE CLAY

In an attempt to compare the stability of a fully consolidated embankment by both total stress and effective stress analyses, a hypothetical embankment was placed on a thirty foot layer of soil having the properties of Boston Blue Clay (BBC). The stress history is assumed as shown in Figure B-1 and embankment geometry as shown in Figure B-2.

All soil properties are those developed for Boston Blue Clay as reported by Azzouz (1977). The normalized strength parameters of BBC are plotted in Figure B-3 for three testing conditions: Plane Strain Compression (PSC), Direct-Simple Shear (DSS) and Plane Strain Extension (PSE). These tests represent three typical failure modes along a failure surface beneath an embankment. Note that the strengths given in Figure B-3 are the shear strengths on the failure surface at failure (τ_{ff}) and not the maximum shear (q_f). The strengths are also corrected for strain compatibility because the peak strength occurs at different strains for the three testing conditions listed above and hence the peak strengths can not occur simultaneously. The average of the peak strengths for the tests is higher than the average strength at any given level of strain. Therefore the values of the Plane Strain Extension and Compression

tests are reduced to represent the strength mobilized at a uniform strain along the failure surface. Figure B-4 shows the undrained strength parameters of BBC used in the STAB3D method of slices. These parameters are also for the shear on the failure surface at failure (τ_{ff}) and corrected for strain compatibility. Figures B-3 and B-4 are tabulated in Table B-1.

The 'end-of-construction' class of stability was evaluated to check that stability would be a problem and to compare the results of Simplified Bishop (1955) and STAB3D (Azzouz, 1977) methods of slices. The Simplified Bishop method of analysis used two different strength assumptions. The first analysis employed the Direct-Simple Shear (DSS) strength along the entire failure surface as the average value and the second analysis used Plane Strain Compression (PSC), Direct-Simple Shear (DSS) and Plane Strain Extension (PSE) in zones from the center of the embankment to the crest, the crest to the toe and beyond the toe respectively for an Active, Direct and Passive (ADP) analysis (Figure B-2). The STAB3D program considers the inclination of the failure surface at each slice to compute the shear strength based on the anisotropic soil properties input into the program.

The normalized soil parameters and the stress history of the clay were used to obtain the variation in the

undrained strength with depth, SHANSEP approach, as calculated in Table B-3 and plotted in Figure B-5A. The Simplified Bishop method of analysis with the DSS strength for an average strength along the entire failure surface resulted in a factor of safety of 1.02 and failure surface shown in Figure B-6. Using the same method but with three zones for strength also resulted in a factor of safety of 1.02. However, in the process of finding the minimum factor of safety the failure surface has shifted away from the PSC zone of higher strength and the agreement in the factor of safety is fortuitous. The shift away from the zone of higher strength is counteracted by the reduction in the driving moment causing failure as a result of the shift. The STAB3D method for the same failure surfaces obtained from the Simplified Bishop method resulted in factors of safety about seven percent lower for the end-of-construction class of stability. The major difference in the two methods is the strength in the zone beneath the slope where the DSS strength is significantly higher than the strength determined using the anisotrophic properties in the STAB3D method (Figure B-6).

The hypothetical embankment was constructed at a rate which allowed for an increase in strength due to consolidation. Settlements computed from one-dimensional analysis were included to obtain the deformed geometry after consolidation

(Table B-2). The embankment crest was assumed to remain level by additional fill to compensate for the settlement. The deformed geometry shown in Figure B-2 was used for all of the long term analyses.

The stability after consolidation was computed using three methods: TSA by Simplified Bishop with three zones for Active, Direct and Passive (ADP) analysis; TSA by the STAB3D method; and as ESA by Simplified Bishop method of slices. The effective stress analysis is the 'type' of analysis typically used for "CD" long term stability problems with drained strength parameters, $\bar{c}$ and $\bar{\phi}$, and hydrostatic pore pressures. For this study the value of cohesion $\bar{c}$ was taken as zero and the friction angle $\bar{\phi}$ as thirty degrees which represents the lower bound for BBC as reported by Azzouz (1977). The ESA factor of safety computed for the embankment was 2.40 and gave a very shallow failure surface (Figure B-7).

The total stress analysis for the long term is the same as a "CU" class stability problem with one-hundred percent consolidation. The consolidation stresses were estimated using the average overburden for the three zones shown in Figure B-2. The zone from the centerline to the crest (crest) consolidated under a load of 1875 PSF = 15 Ft x 115 PCF, the zone from crest to toe (slope) consolidated under a load of 938 PSF = 7.5 Ft x 115 PCF, and no load

AD-A070 839 AIR FORCE INST OF TECH WRIGHT-PATTERSON AFB OH F/G 8/13
TOTAL STRESS VERSUS EFFECTIVE STRESS STABILITY ANALYSIS OF EMBA--ETC(U)
DEC 78 S C BOYCE
UNCLASSIFIED AFIT-CI-79-99T NL

2 OF 2
AD
A070839

END
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FILMED
8-79
DDC

was considered beyond the toe. This assumption compares very well with elastic solutions for embankments on a finite layer. Using the consolidation stresses computed in this manner, the variation in undrained strength with depth was calculated in Table B-4 and B-5 and is shown in Figure B-5B. The undrained strength calculated in this way is conservative because the rotation of the principal stress is not considered in determining the strength. Ladd and Edgers (1972) used $\overline{\text{CAU}}$ DSS tests to show that the rotation of the principal stress caused an increase in the strength compared to $\overline{\text{CK}_0\text{U}}$ DSS. The total stress analysis factor of safety should also be smaller than for the ESA due to the pore pressures generated during shear to failure. The factor of safety from the ADP analysis was 1.40 and from the STAB3D analysis was 1.14 with failure surfaces shown in Figure B-7. The largest differences in the shear strength along the failure surface for the two total stress analyses is again in the slope zone where the DSS strengths are higher than those calculated from STAB3D anisotropic parameters.

TABLE B-1

UNDRAINED STRENGTH RATIOS CORRECTED FOR STRAIN COMPATIBILITY vs OCR FOR BOSTON BLUE CLAY

(Azzouz 1977)

OCR	$\frac{\tau_{ff}(PSC)}{\bar{\sigma}_{vc}}$	$\frac{\tau_{ff}(DSS)}{\bar{\sigma}_{vc}}$	$\frac{\tau_{ff}(PSE)}{\bar{\sigma}_{vc}}$	K_s	b/a
1	0.263	0.200	0.137	0.52	0.88
2	0.435	0.365	0.275	0.63	0.96
4	0.725	0.605	0.504	0.70	0.93
6	0.956	0.800	0.692	0.72	0.91
8	1.190	0.970	0.864	0.73	0.89
12	1.594	1.236	1.203	0.76	0.83
16	1.830	1.440	1.446	0.79	0.83

SETTLEMENT AT THE CENTERLINE

BBC MODEL EMBANKMENT

DEPTH (Ft)	$\bar{\sigma}_{vo}$ (KSF)	$\bar{\sigma}_{vm}$ (KSF)	$\bar{\sigma}_{vf}$ (KSF)	$RR \log \frac{\bar{\sigma}_{vm}}{\bar{\sigma}_{vo}}$	$CR \log \frac{\bar{\sigma}_{vf}}{\bar{\sigma}_{vm}}$	ρ_{cf} (Ft)
-2.5	0.144	4.375	2.019	.0516	—	.258
-7.5	0.432	3.125	2.307	.0364	—	.182
-12.5	0.708	1.875	2.583	.0211	.0348	.279
-17.5	0.971	1.375	2.846	.0076	.0790	.433
-22.5	1.234	1.625	3.109	.0060	.0704	.382
-27.5	1.497	1.875	3.372	.0049	.0637	.343

Σ 1.877'

TABLE B-2

EL. (FEET)	$\bar{\sigma}_{vo}$ (KSF)	$\bar{\sigma}_{vm}$ (KSF)	OCR	$S_u/\bar{\sigma}_{vo}$ DSS	S_u DSS (PSF)	$S_u/\bar{\sigma}_{vo}$ PSE	S_u PSE (PSF)
97.5	0.144	4.375	30.38	1.900	274	1.970	284
92.5	0.432	3.125	7.23	0.905	391	0.798	345
87.5	0.708	1.875	2.65	0.445	315	0.350	249
82.5	0.971	1.375	1.42	0.269	261	0.192	186
77.5	1.234	1.625	1.32	0.254	313	0.180	222
72.5	1.497	1.875	1.25	0.242	362	0.170	254

EL. (FEET)	$S_u/\bar{\sigma}_{vo}$ PSC	S_u PSC (PSF)	K_s	b/a	a (PSF)	d	e
97.5	2.410	347	0.84	0.78	319	0.392	0.087
92.5	1.100	475	0.73	0.90	411	0.190	0.156
87.5	0.530	375	0.65	0.95	309	0.098	0.212
82.5	0.333	323	0.58	0.93	261	0.135	0.266
77.5	0.314	387	0.57	0.92	319	0.154	0.274
72.5	0.302	452	0.56	0.91	362	0.172	0.282

$a = \frac{1}{2}(S_u(V) + S_u(H)) = \frac{1}{2}\tau_{ff}(V)(1+K_s)$

$d = 1 - (b/a)^2$

$e = (1-K_s)/(1+K_s)$

UNDRAINED STRENGTH BEFORE CONSOLIDATION

BBC MODEL EMBANKMENT

TABLE B-3

EL. (FEET)	$\bar{\sigma}_{vf}$ (KSF)	$\bar{\sigma}_{vm}$ (KSF)	OCR	$S_u/\bar{\sigma}_{vf}$ DSS	S_u DSS (PSF)	$S_u/\bar{\sigma}_{vf}$ PSC	S_u PSC (PSF)
97.5	1.082	4.375	4.04	0.610	660	0.730	790
92.5	1.370	3.125	2.28	0.400	536	0.470	644
87.5	1.645	1.875	1.14	0.225	360	0.290	477
82.5	1.908	1.908	1.00	0.200	382	0.263	502
77.5	2.172	2.172	1.00	0.200	434	0.263	571
72.5	2.435	2.435	1.00	0.200	487	0.263	640

EL. (FEET)	K_s	b/a	a (PSF)	d	e
97.5	0.70	0.93	672	0.135	0.176
92.5	0.65	0.96	531	0.078	0.212
87.5	0.54	0.90	367	0.190	0.299
82.5	0.52	0.88	382	0.226	0.316
77.5	0.52	0.88	434	0.226	0.316
72.5	0.52	0.88	486	0.226	0.316

$a = \frac{1}{2}(S_u(V) + S_u(H)) = \frac{1}{2}\tau_{ff}(V)(1+K_s)$
$d = 1-(b/a)^2$
$e = (1-K_s)/(1+K_s)$

UNDRAINED STRENGTH AFTER CONSOLIDATION

FOR SLOPE OF BBC MODEL EMBANKMENT

TABLE B-4

EL. (FEET)	$\bar{\sigma}_{vf}$ (KSF)	$\bar{\sigma}_{vm}$ (KSF)	OCR	$S_u/\bar{\sigma}_{vf}$ PSC	S_u PSC (PSF)
97.5	2.019	4.375	2.17	0.460	929
92.5	2.307	3.125	1.35	0.320	738
87.5	2.583	2.583	1.00	0.263	679
82.5	2.846	2.846	1.00	0.263	748
77.5	3.109	3.109	1.00	0.263	818
72.5	3.372	3.372	1.00	0.263	887

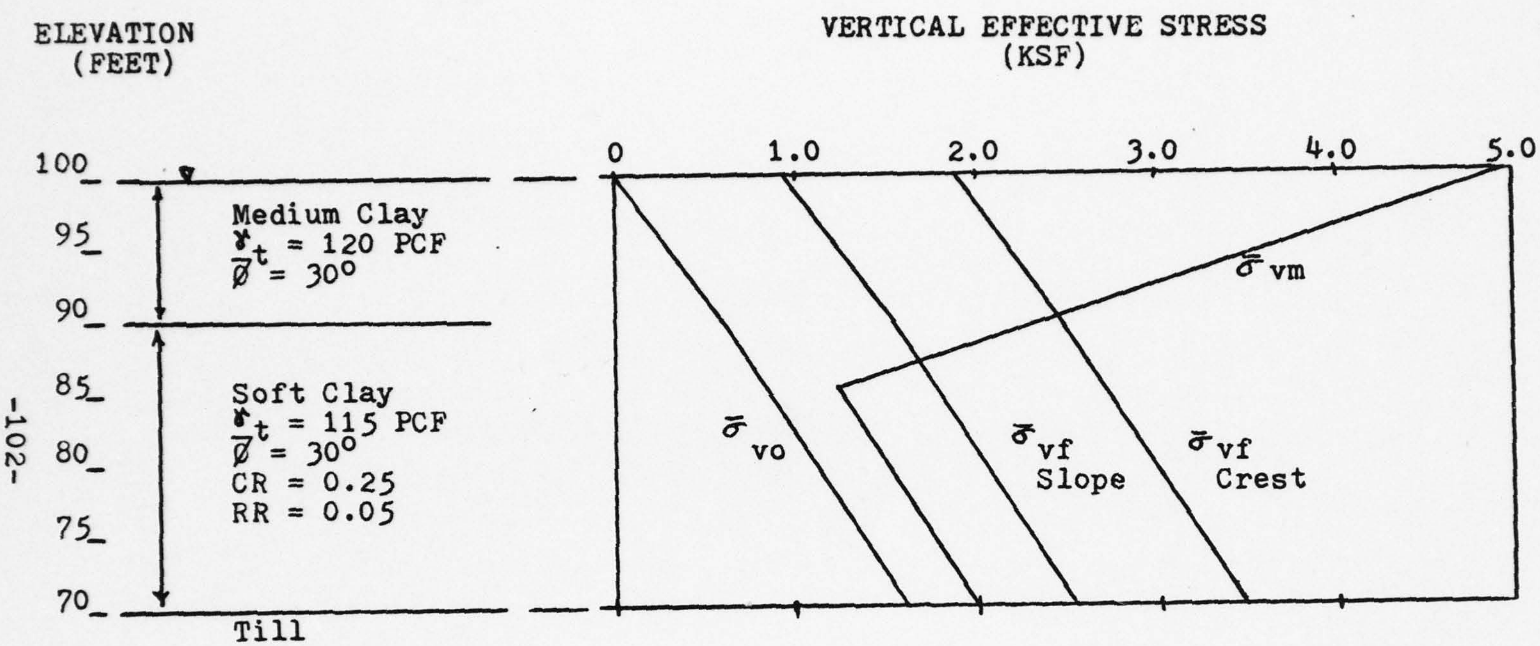
EL. (FEET)	K_s	b/a	a (PSF)	d	e
97.5	0.64	0.96	762	0.078	0.220
92.5	0.57	0.92	579	0.154	0.274
87.5	0.52	0.88	516	0.226	0.316
82.5	0.52	0.88	568	0.226	0.316
77.5	0.52	0.88	622	0.226	0.316
72.5	0.52	0.88	674	0.226	0.316

$$a = \tfrac{1}{2}(S_u(V) + S_u(H)) = \tfrac{1}{2}\tau_{ff}(V)(1+K_s)$$
$$d = 1-(b/a)^2$$
$$e = (1-K_s)/(1+K_s)$$

UNDRAINED STRENGTH AFTER CONSOLIDATION

FOR CREST OF BBC MODEL EMBANKMENT

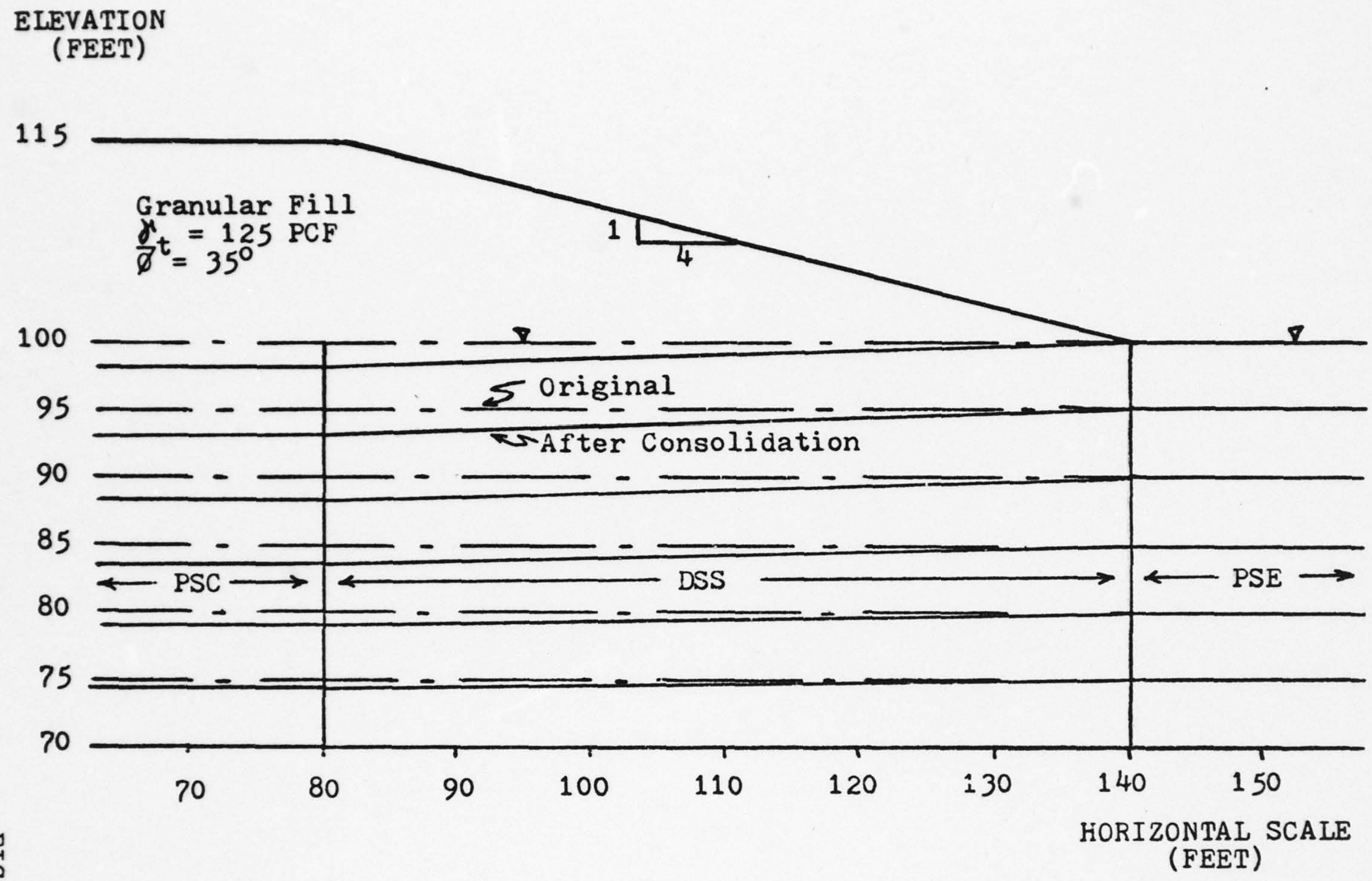
TABLE B-5



STRESS HISTORY

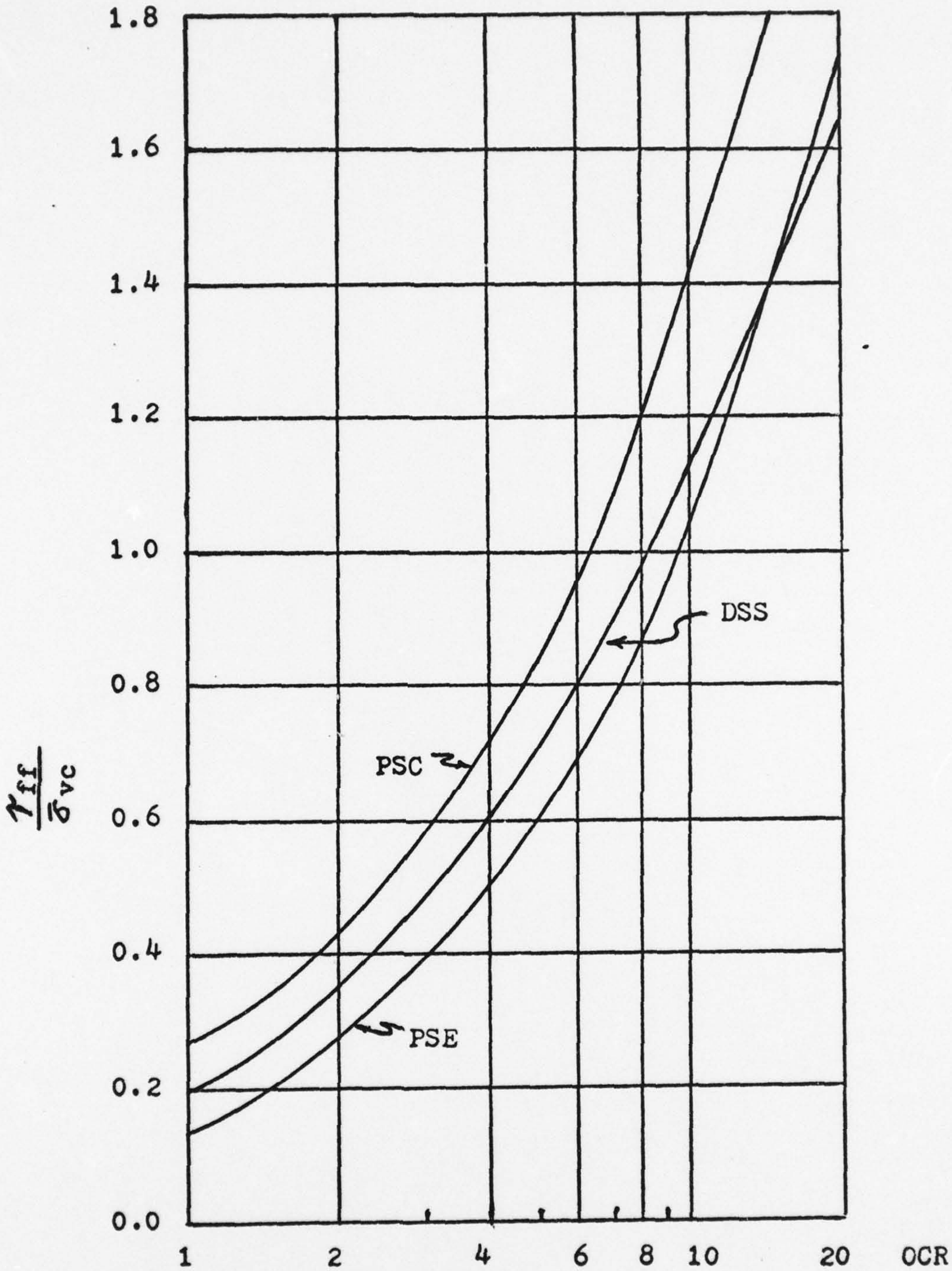
BBC MODEL EMBANKMENT

FIGURE B-1



EMBANKMENT GEOMETRY
BBC MODEL EMBANKMENT

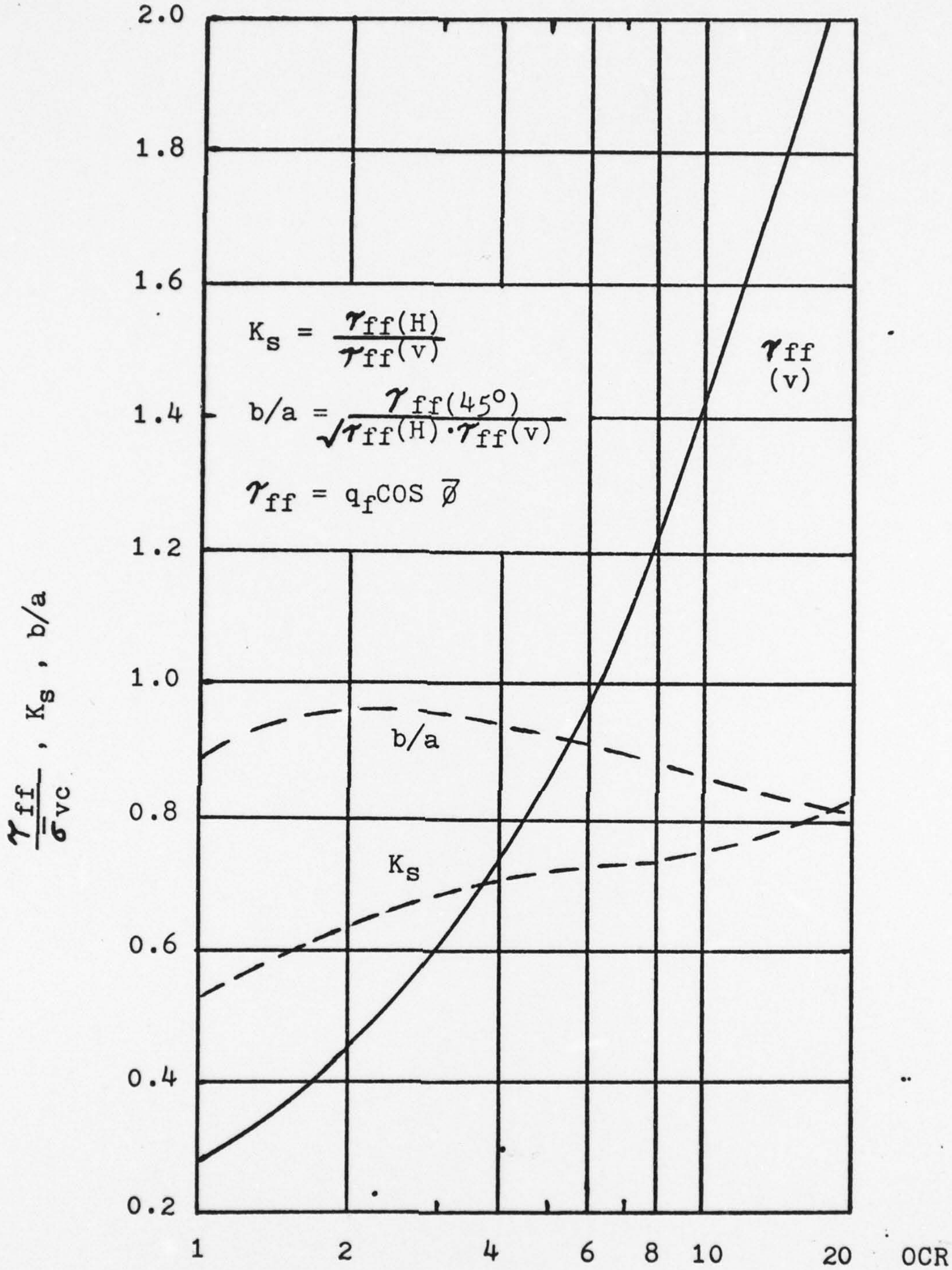
FIGURE B-2



UNDRAINED STRENGTH
BOSTON BLUE CLAY

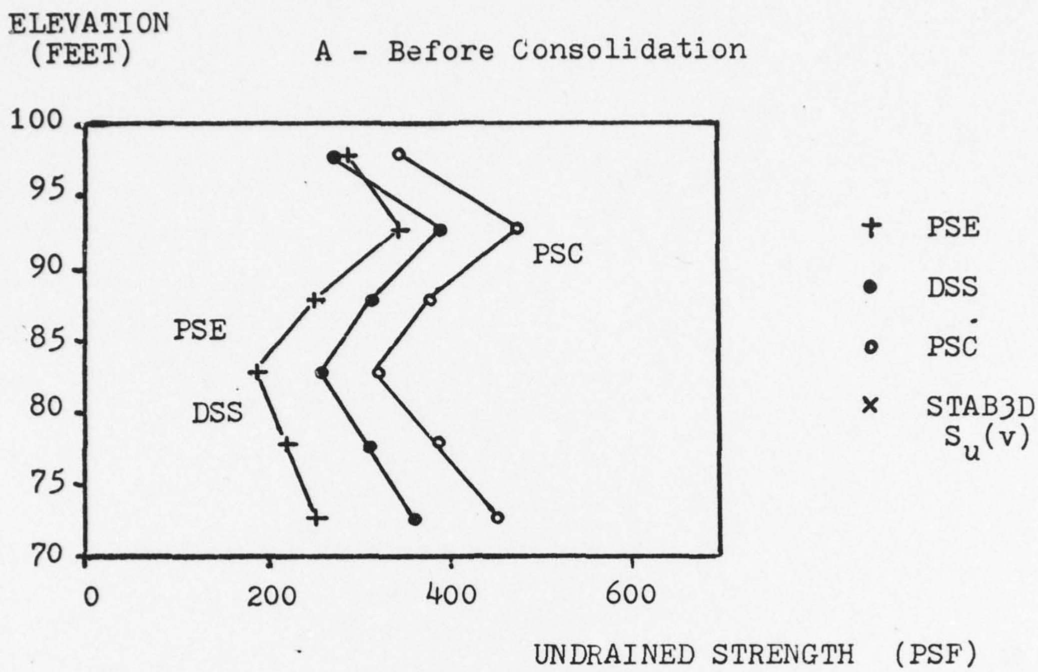
(Azzouz 1977)

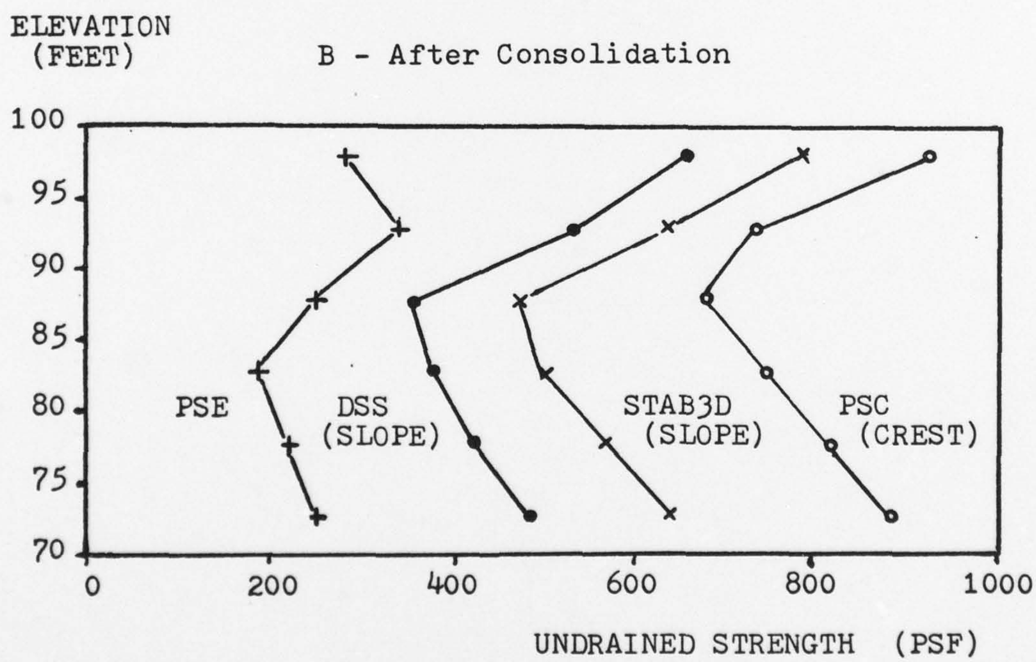
FIGURE B-3



ANISOTROPIC UNDRAINED STRENGTH PARAMETERS
BOSTON BLUE CLAY
(Azzouz, 1977)

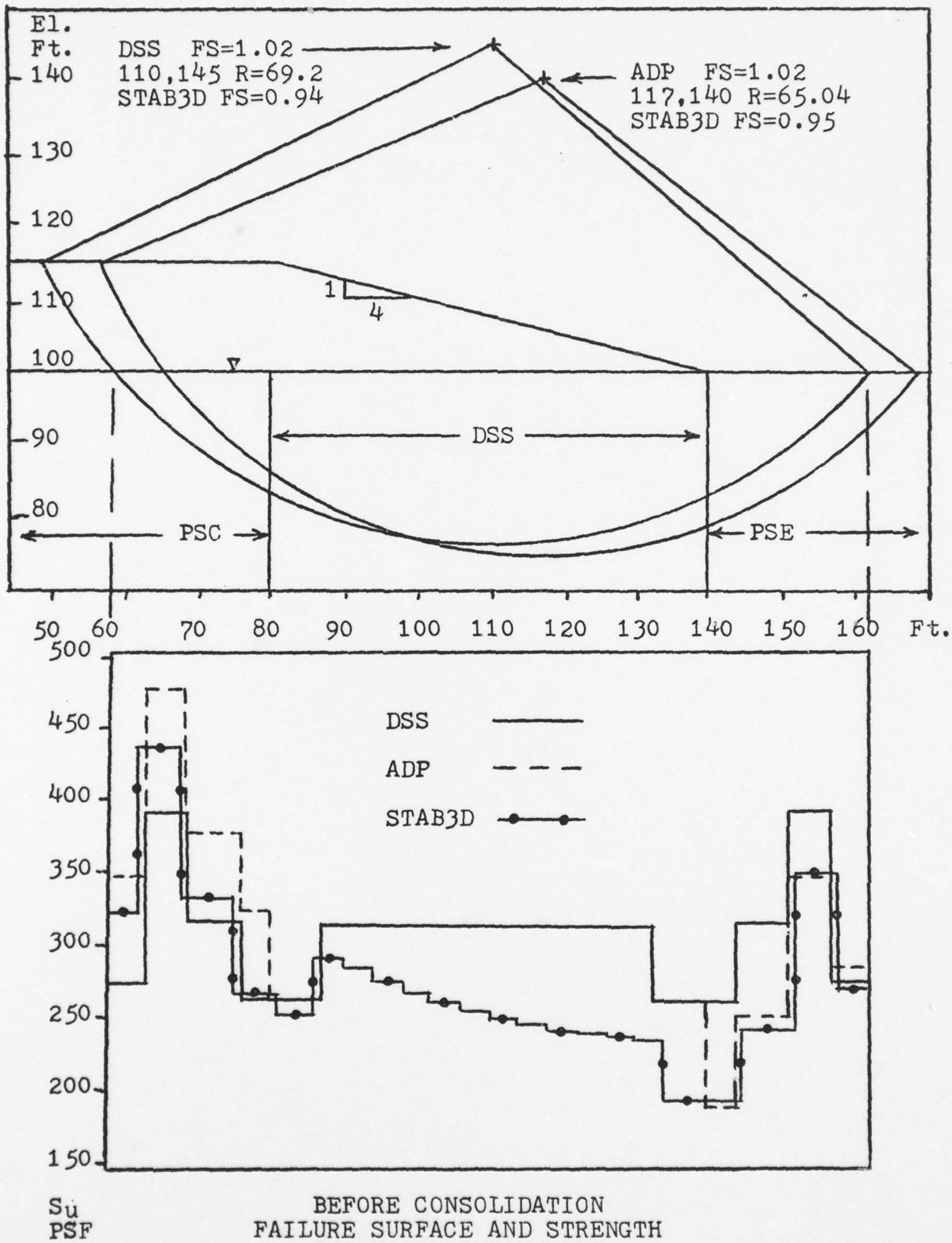
FIGURE B-





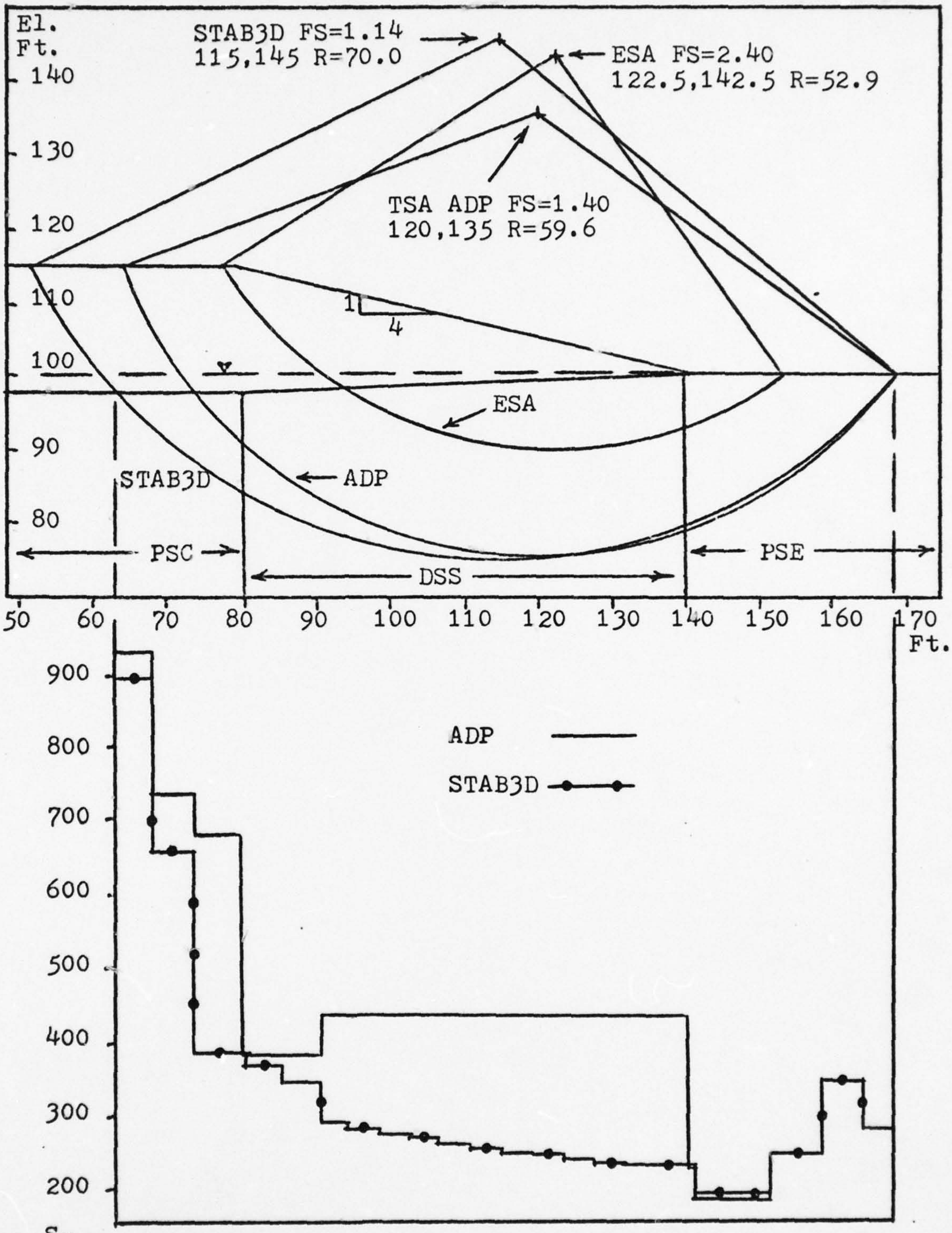
UNDRAINED STRENGTH
BBC MODEL EMBANKMENT

FIGURE B-5



BEFORE CONSOLIDATION
FAILURE SURFACE AND STRENGTH
BBC MODEL EMBANKMENT

FIGURE B-6



AFTER CONSOLIDATION FAILURE SURFACE
AND STRENGTH BBC MODEL EMBANKMENT

FIGURE B-7